Document 00911

NOTICE OF
ADDENDUM NO. 8

Date of Addendum: 4/8/15

PROJECT NAME: Gillette Trunkline (Genesee Segment) Drainage & Paving

PROJECT NO: WBS No. M-410290-0003-4

BID DATE: **April 23, 2015** (Change in Bid Date)

FROM: Ravi Kaleyatodi, P.E., CPM, Senior Assistant Director
City of Houston, Public Works and Engineering Department
611 Walker St., 15th Floor
Houston, Texas 77002
Attn: Ellen Maas, P.E., Project Manager

TO: Prospective Bidders

The referenced Addendum forms a part of the Bidding Documents and will be incorporated into the Contract documents, as applicable.

Written questions regarding this Addendum may be submitted to the Project Manager following the procedures specified in Document 00200 – Instructions to Bidders. Immediately notify the City Engineer through the named Project Manager upon finding discrepancies or omissions in the Bid Documents.

This Addendum includes:

ADDENDUM SYNOPSIS

Change in Bid Date
Changes to Project Manual

Ravi Kaleyatodi

Ravi Kaleyatodi, P.E., CPM
Senior Assistant Director
Engineering Branch
Engineering and Construction Division

DATED: 4/8/15

END OF DOCUMENT

Document 00910

ADDENDUM NO. 8

Date of Addendum: 9/8/15

PROJECT NAME: Genesee Street Drainage and Paving Improvements

PROJECT NO: WBS No. M-410290-0003-4

BID DATE: **April 23, 2015** (Change in Bid Date)

FROM: James T. Lincoln, P.E., City Engineer
City of Houston, Department of Public Works and Engineering
611 Walker Street, 15th Floor
Houston, Texas 77002
Attn: Ellen Maas, P.E., Project Manager

TO: Prospective Bidders

This Addendum forms a part of the Bidding Documents and will be incorporated into the Contract documents, as applicable. Insofar as the original Project Manual and Drawings are inconsistent, this Addendum governs.

CHANGE IN BID DATE

The Bid Date for this Project has been changed from April 9, 2015 to April 23, 2015. Time of day and place for submittal of bid remains the same.

CHANGES TO PROJECT MANUAL

BIDDING REQUIREMENTS

1. Document 00320 – Geotechnical Information. Replace entire document. Added Supplemental Geotechnical Investigation for Storm Sewer Tunnels at W. Gray and Welch.

END OF ADDENDUM NO. 8

DATED: 4/8/15

Ravi Kaleyatodi, P.E., CPM
Senior Assistant Director
Department of Public Works and Engineering

END OF DOCUMENT

STATE OF TEXAS
STEPHEN A. SPARKS
66994
REGISTERED
PROFESSIONAL ENGINEER

Document 00320

GEOTECHNICAL INFORMATION

1. DOCUMENT INCLUDES

   A. Soils investigation reports.
   B. Bidder's responsibilities.

2. RELATED DOCUMENTS

   A. Document 00340 – Environmental Information
   B. Section 02260 - Trench Safety Systems

3. SITE INVESTIGATION REPORTS

   A. In the design and preparation of Contract documents for this Project, the City and Design Consultant have used information in geotechnical reports for the investigation and analysis of soils and subsurface conditions at the Project site.

   B. An electronic copy of the report for this project is included in a CD-Rom affixed to the inside front cover of the project manual.

   C. Neither the City nor Design Consultant is responsible for accuracy or completeness of any information or data.

4. GEOTECHNICAL REPORTS

   A. Report No. G166-12B – R1 Prepared by (Firm Name): Aviles Engineering Corp. Title: Geotechnical Investigation City of Houston Gillette Trunkline (Genesee Segment) Drainage and Paving Improvements COH WBS No. M-410290-0003-3 Report Date: September 2014 No. of Pages: 79

   B. Report No. G166-12B – SUPPLEMENTAL Prepared by (Firm Name): Aviles Engineering Corp. Title: Geotechnical Investigation Gillette Trunkline (Genesee Segment) Drainage and Paving Improvements Storm Sewer Tunnels at W. Gray and Welch WBS No. M-410290-0003-3 Houston, Texas Report Date: March 2015 No. of Pages: 60

5. BIDDER RESPONSIBILITIES

   A. Bidder shall take full responsibility for interpretation and use of information contained in above listed reports for its bidding and construction purposes.

   B. Bidder may perform additional soils investigations as Bidder deems appropriate.

END OF DOCUMENT

ENGINEERING CORP.

GEOTECHNICAL INVESTIGATION

GILLETTE TRUNKLINE (GENESEE SEGMENT)
DRAINAGE AND PAVING IMPROVEMENTS
STORM SEWER TUNNELS AT W. GRAY AND WELCH
WBS NO. M-410290-0003-3
HOUSTON, TEXAS

Reported to:
HR Green, Inc.
Houston, Texas

by

Aviles Engineering Corporation
5790 Windfern
Houston, Texas 77041
713-895-7645

REPORT NO. G166-12B - SUPPLEMENTAL

March 2015

Addendum No. 8

AVILES
ENGINEERING CORP.

5790 Windfern Road
Houston, Texas 77041
Tel: (713)-895-7645
Fax: (713)-895-7943

March 20, 2015

Ms. Celeste Jain, P.E.
Project Engineer
HR Green, Inc.
11011 Richmond Avenue, Suite 375
Houston, Texas 77042

Reference: **Geotechnical Investigation**
City of Houston Gillette Trunkline (Genesee Segment)
Drainage and Paving Improvements
Storm Sewer Tunnels at W. Gray and Welch
Houston, Texas
WBS No. M-410290-0003-3
AEC Report No. G166-12B - Supplemental

Dear Ms. Jain,

Aviles Engineering Corporation (AEC) is pleased to present this supplemental report of the results of our geotechnical investigation for the above referenced project. Notice to proceed for the geotechnical investigation was provided via email on January 14, 2015 by Ms. Celeste Jain, P.E., of HR Green, based on AECs proposal G2014-11-17R1, dated December 9, 2014.

AEC appreciates the opportunity to be of service to you. Please call us if you have any questions or comments concerning this report or when we can be of further assistance.

Respectfully submitted,
Aviles Engineering Corporation
(TBPE Firm Registration No. F-42)

3/20/2015
STATE OF TEXAS
WILBER L. WANG
99226
LICENSED
PROFESSIONAL ENGINEER

Wilber L. Wang, M.Eng., P.E.
Project Engineer

Shou Ting Hu, M.S.C.E., P.E.
Principal Engineer

Reports Submitted: 3 HR Green, Inc.
1 File (electronic)

Z:\ENGINEERING\REPORTS\2012\166-12 COH MONTROSE AREA & MIDTOWN STORM SEWER IMPROVEMENTS - HR GREEN, INC. (WILBER)\G166-12 SEGMENT B SUPPLEMENTAL FINAL.DOCX

AVILES ENGINEERING CORP.

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AVILES ENGINEERING CORP.

EXECUTIVE SUMMARY

The supplemental report submitted herein presents the results of Aviles Engineering Corporation's (AEC) geotechnical investigation for the City of Houston's (COH) proposed Reinforced Concrete Box (RCB) Storm Sewers along Genesee Street that will be installed by tunnel method at the intersection of W. Gray Street and Welch Street, for the Gillette Trunkline (Genesee Segment) Drainage and Paving Improvements project, in Houston, Texas (Houston Key Map 493P). Based on plan and profile drawings (dated October 15, 2014) provided to AEC by HR Green for the Genesee Segment, two sections of 10 foot by 10 foot RCB storm sewers will be installed by tunnel method along Genesee Street. A 150 foot long tunnel will be at the intersection of W. Gray Street and an 82 foot long tunnel will be at the intersection of Welch Street. The invert depth of the tunnel at the Welch Street intersection is approximately 23.4 feet, while the invert depth at the W. Gray Street intersection is approximately 25 feet deep.

1. Subsurface Soil Conditions: A generalized subsurface profile along the storm sewer alignment is presented on Plate B-1, in Appendix B.

   W. Gray Tunnel: Based on Borings B-8A and B-8B, the subsurface soil conditions at the W. Gray tunnel generally consist of stiff to hard fat/lean clay (CH/CL) from the ground surface to the boring termination depths of 40 feet. An approximately 5 foot thick clayey sand (SC) layer was encountered at a depth of 23 feet in Boring B-8A.

   Welch Tunnel: Based on Boring B-10A, the subsurface soil conditions at the Welch tunnel generally consist of stiff to hard fat/lean clay (CH/CL) from the ground surface to the boring termination depth of 40 feet.

2. Subsurface Soil Properties: The subsurface clayey soils (i.e. not including clayey sand) have high to very high plasticity, with liquid limits (LL) ranging from 38 to 86, and plasticity indices (PI) ranging from 24 to 51. The cohesive soils encountered are classified as "CL" and "CH" type soils and granular soils were classified as "SC" in accordance with ASTM D 2487.

3. Groundwater Conditions: Groundwater was encountered at a depth of 20 to 28 feet below grade during drilling in Borings B-8A and B-8B and was subsequently observed at a depth of 17.6 to 23.2 feet drilling was complete. Groundwater was not encountered in Boring B-10A during drilling. Groundwater along the alignment may be pressurized. After completion of drilling, Borings B-8B and B-10A were converted to piezometers. A detailed description of ground water readings is presented on Table 3 in Section 4.1 of this report.

4. Hazardous Materials: No signs of visual staining or odors were encountered during field drilling or during processing of the soil samples in the laboratory.

5. Design parameters and recommendations for installation of storm sewers by tunnel method are presented in Section 5.2 of this report.

6. Design parameters and recommendations for concrete pavement are presented in Section 5.4 of this report.

This Executive Summary should not be used without the full text of this report.

AVILES ENGINEERING CORP.

GEOTECHNICAL INVESTIGATION

GILLETTE TRUNKLINE (GENESEE SEGMENT)
DRAINAGE AND PAVING IMPROVEMENTS
STORM SEWER TUNNELS AT W. GRAY AND WELCH
WBS NO. M-410290-0003-3
HOUSTON, TEXAS

1.0 INTRODUCTION

1.1 General

The supplemental report submitted herein presents the results of Aviles Engineering Corporation's (AEC) geotechnical investigation for the City of Houston's (COH) proposed Reinforced Concrete Box (RCB) Storm Sewers along Genesee Street that will be installed by tunnel method at the intersection of W. Gray Street and Welch Street, for the Gillette Trunkline (Genesee Segment) Drainage and Paving Improvements project, in Houston, Texas (Houston Key Map 493P). A vicinity map is presented on Plate A-1, in Appendix A. This supplemental report is for the storm sewer tunnels at W. Gray and Welch Street only, and should be used in combination with AEC's geotechnical report for the Gillette Trunkline (Genesee Segment), AEC Report G166-12B R1, dated September 18, 2014.

Based on plan and profile drawings (dated October 15, 2014) provided to AEC by HR Green for the Genesee Segment, two sections of 10 foot by 10 foot RCB storm sewers will be installed by tunnel method along Genesee Street. A 150 foot long tunnel will be at the intersection of W. Gray Street and an 82 foot long tunnel will be at the intersection of Welch Street. The invert depth of the tunnel at the Welch Street intersection is approximately 23.4 feet, while the invert depth at the W. Gray Street intersection is approximately 25 feet deep.

1.2 Purpose and Scope

The purpose of this geotechnical investigation is to evaluate the subsurface soil conditions along the alignment and develop geotechnical engineering recommendations for design and construction of storm sewers by tunnel method. The scope of this geotechnical investigation is summarized below:

1. Drilling and sampling three geotechnical borings to 40 feet below existing grade;
2. Soil laboratory testing on selected soil samples;
3. Engineering analyses and recommendations for installation of storm sewers by tunnel method, including tunnel access shafts, reaction walls, and tunnel stability;
4. Construction recommendations for installation of storm sewers by tunnel method.

2.0 SUBSURFACE EXPLORATION

2.1 Soil Borings

The boring layout and depths were selected by AEC in general accordance with Chapter 11 of the COH Infrastructure Design Manual (IDM), based on plan and profile drawings (dated October 15, 2014) provided by HR Green.

The subsurface exploration consisted of drilling and sampling a total of three soil borings (Borings B-8A, B-8B, and B-10A) to 40 feet below existing grade. Borings B-5 through B-12 were performed along the Genesse Street alignment between W. Dallas Street and Tuam Street, and are presented in AEC Report G166-12B R1. The boring locations are shown on the Boring Location Plan on Plate A-2, in Appendix A. Total drilling footage is 120 feet. Boring survey data was provided to AEC and is included on the boring logs. The boring designations and depths and corresponding storm sewer tunnel invert depths are presented in Table 1 below.

Table 1. Boring Number, Station, and Tunnel Invert Depth

Boring No.	Boring Depth (ft)	Station No./Alignment	Invert Depth near Boring (ft)	Piezometer Depth (ft)
B-8A	40	19+09.40 (Genesse)	24.9	-
B-8B	40	20+60.49 (Genesse)	25.1	30
B-10A	40	8+20.91 (Genesse)	23.4	30

The field drilling was performed with a truck-mounted drilling rig primarily using dry auger method, and then using wet rotary method once water-bearing granular soils were encountered or the borings began to cave in. Undisturbed samples of cohesive soils were obtained from the borings by pushing 3-inch diameter thin-wall, seamless steel Shelby tube samplers in general accordance with ASTM D 1587. Strength of the cohesive soils was estimated in the field using a hand penetrometer. The undisturbed samples of cohesive soils were extruded mechanically from the core barrels in the field and wrapped in aluminum foil; all samples were sealed in plastic bags to reduce moisture loss and disturbance. The samples were then placed in core boxes and transported to the AEC laboratory for testing and further study. Borings B-8B and B-10A were converted to piezometers upon completion of drilling. Boring B-8A was grouted with cement-bentonite upon completion of drilling and the existing pavement was patched with asphalt.

3.0 LABORATORY TESTING PROGRAM

Soil laboratory testing was performed by AEC personnel. Samples from the borings were examined and classified in the laboratory by a technician under the supervision of a geotechnical engineer. Laboratory tests were performed on selected soil samples in order to evaluate the engineering properties of the foundation soils in accordance with applicable ASTM Standards. Atterberg limits, moisture contents, percent passing a No. 200 sieve, and dry unit weight tests were performed on typical samples to establish the index properties and confirm field classification of the subsurface soils. Strength properties of cohesive soils were determined by means of unconfined compression (UC) and undrained-unconsolidated (UU) triaxial tests performed on undisturbed samples. The test results are presented on the boring logs. Details of the soils encountered in the borings are presented on Plates A-3 through A-5, in Appendix A. A key to the boring logs, classification of soils for engineering purposes, terms used on boring logs, and reference ASTM Standards for laboratory testing are presented on Plates A-6 through A-9, in Appendix A. A summary of the lab data is presented on Plates A-10 and A-11, in Appendix A.

4.0 SITE CONDITIONS

Based on our site visit, Genesee Street between West Dallas Street and West Gray Street is a one-way roadway and between West Gray Street and Tuam Street is a narrow two lane (one lane in each direction) roadway. A summary of pavement types encountered in our borings is presented on Table 2.

Table 2. Existing Pavement Encountered at Pavement Borings

Boring No.	Street	Pavement Section
B-8A	Genesee	2" asphalt, 7" sand, shell, and gravel
B-8B	Genesee	8.5" concrete, 3.5" cement stabilized sand and gravel
B-10A	Genesee	4" asphalt, 6" sand and gravel

4.1 Subsurface Conditions

A generalized subsurface profile along the storm sewer alignment is presented on Plate B-1, in Appendix B. Soil strata encountered in our borings are summarized below:

Boring	Depth (ft)	Description of Stratum
B-8A	0 - 0.2	Pavement: 2" asphalt
	0.2 - 0.8	Base: 7" sand, shell, and gravel
	0.8 - 4	Very stiff, Fat Clay w/Sand (CH)
	4 - 10	Stiff to very stiff, Fat Clay (CH), with slickensides
	10 - 16	Very stiff to hard, Lean Clay (CL), with abundant silt partings
	16 - 18	Very stiff, Fat Clay (CH)
	18 - 28	Clayey Sand (SC)
	28 - 40	Hard, Fat Clay (CH), with slickensides
B-8B	0 - 0.7	Pavement: 8.5" concrete
	0.7 - 1	Base: 3.5" cement stabilized sand and gravel
	1 - 2	Fill: very stiff, Fat Clay (CH), with lime stabilized clay seams and strong organic odor
	2 - 10	Stiff to very stiff, Fat Clay (CH)
	10 - 14	Very stiff to hard, Lean Clay (CL)
	14 - 22	Stiff to hard, Fat Clay (CH), with slickensides
	22 - 40	Stiff to hard, Lean Clay (CL), with slickensides and fat clay seams
B-10A	0 - 0.3	Pavement: 4" asphalt
	0.3 - 0.8	Base: 6" sand and gravel
	0.8 - 8	Stiff to very stiff, Fat Clay (CH)
	8 - 14	Very stiff to hard, Lean Clay w/Sand (CL)
	14 - 22	Stiff to very stiff, Fat Clay w/Sand (CH), with slickensides
	22 - 37	Very stiff to hard, Fat Clay (CH), with slickensides
	37 - 40	Hard, Lean Clay (CL), with silt partings

<u>Subsurface Soil Properties:</u> The subsurface clayey soils (i.e. not including clayey sand) have high to very high plasticity, with liquid limits (LL) ranging from 38 to 86, and plasticity indices (PI) ranging from 24 to 51. The cohesive soils encountered are classified as "CL" and "CH" type soils and granular soils were classified as "SC" in accordance with ASTM D 2487. High plasticity clays can undergo significant volume changes due to seasonal changes in moisture contents. "CH" soils undergo significant volume changes due to seasonal changes in soil moisture contents. "CL" type soils with lower LL (less than 40) and PI (less than 20) generally do not undergo significant volume changes with changes in moisture content. However, "CL" soils with LL approaching 50 and PI greater than 20 essentially behave as "CH" soils and could undergo significant volume changes. Slickensides were encountered in the fat clays.

Groundwater Conditions: Groundwater was encountered at a depth of 20 to 28 feet below grade during drilling in Borings B-8A and B-8B and was subsequently observed at a depth of 17.6 to 23.2 feet drilling was complete. Groundwater was not encountered in Boring B-10A during drilling. Groundwater along the alignment may be pressurized. After completion of drilling, Borings B-8B and B-10A were converted to piezometers. Piezometer installation details are presented on Plates B-2 and B-3, in Appendix B. Detailed groundwater levels are summarized in Table 3. Piezometer installation and plugging reports are presented in Appendix E.

Table 3. Groundwater Depths below Existing Ground Surface

Boring No.	Date Drilled	Boring Depth (ft)	Groundwater Depth in Boring (ft)	Groundwater Depth in Piezometer (ft)
B-8A	1/27/15	40	28 (Drilling) 23.2 (1/4 Hr)	-
B-8B/PZ-2A	1/28/15	40	20 (Drilling) 17.6 (1/4 Hr)	5.7 (3/2/15) 5.2 (3/20/15)
B-10A/PZ-3A	1/27/15	40	Dry (Drilling)	29.3 (1/28/15) 26.0 (3/2/15) 5.2 (3/20/15)

The information in this report summarizes conditions found on the dates the borings were drilled. It should be noted that our groundwater observations are short-term; groundwater depths and subsurface soil moisture contents will vary with environmental variations such as frequency and magnitude of rainfall and the time of year when construction is in progress.

4.2 Hazardous Materials

No signs of visual staining or odors were encountered during field drilling or during processing of the soil samples in the laboratory.

4.3 Subsurface Variations

It should be emphasized that: (i) at any given time, groundwater depths can vary from location to location, and (ii) at any given location, groundwater depths can change with time. Groundwater depths will vary with seasonal rainfall and other climatic/environmental events. Subsurface conditions may vary away from and in between the boring locations.

Clay soils in the Houston area typically have secondary features such as slickensides and contain sand/silt seams/lenses/layers/pockets. It should be noted that the information in the boring logs is based on 3-inch diameter soil samples which were generally obtained continuously at intervals of 2 from the ground surface to a depth of 20 feet in the borings, then at intervals of 5 feet thereafter to the boring termination depths of 40 feet. A detailed description of the soil secondary features may not have been obtained due to the small sample size and sampling interval between the samples. Therefore, while a boring log shows some soil secondary features, it should not be assumed that the features are absent where not indicated on the boring logs.

5.0 GEOTECHNICAL ENGINEERING RECOMMENDATIONS

Based on plan and profile drawings (dated October 15, 2014) provided to AEC by HR Green for the Genesee Segment, two sections of 10 foot by 10 foot RCB storm sewers will be installed by tunnel method along Genesee Street. A 150 foot long tunnel will be at the intersection of W. Gray Street and an 82 foot long tunnel will be at the intersection of Welch Street. The invert depth of the tunnel at the Welch Street intersection is approximately 23.4 feet, while the invert depth at the W. Gray Street intersection is approximately 25 feet deep.

5.1 Geotechnical Parameters for Underground Utilities

Recommended geotechnical parameters for the subsurface soils along the alignment to be used for design of storm sewers are presented on Plate C-1, in Appendix C. The design values are based on the results of field and laboratory test data on individual boring logs as well as our experience. It should be noted that because of the variable nature of soil stratigraphy, soil types and properties along the alignment or at locations away from a particular boring may vary substantially.

5.2 Tunneling and Its Influence on Adjacent Structures

The Contractor is responsible for designing, constructing, implementing, and monitoring safe tunneling excavation and protecting existing structures in the vicinity from adverse effects resulting from construction, and retaining professionals who are qualified and experienced to perform the tasks and who are capable of modifying the system, as required. The following discussion provides general guidelines to the Contractor.

Based on the plan and profile drawings provided by HR Green (dated October 15, 2014), the proposed 10 by 10 foot RCB storm sewer will be installed by tunnel method where the alignment crosses beneath W. Gray and Welch; the alignment stations, approximate tunnel invert depths, and possible subsurface conditions are summarized in Table 5 below.

Table 5. Subsurface Conditions in Borings within Tunnel Zones

Soil Boring	Station	Tunnel Segment	Tunnel Invert Depth (ft)	Soil Types Encountered within Tunnel Zone	Ground Water Depth below Existing Ground Surface (ft)	
					Boring	In Piezometer
B-8A	19+09	W. Gray	24.9	5' - 10': Stiff to very stiff CH 10' - 16': Very stiff to hard CL 16' - 18': Very stiff CH 18' - 28': SC 28' - 35': Hard CH	28 (Drilling) 23.2 (1/4 Hr)	-
B-8B	20+60		25.1	5' - 10': Stiff to very stiff CH 10' - 14': Very stiff to hard CL 14' - 22': Stiff to hard CH 22' - 35': Stiff to hard CL	20 (Drilling) 17.6 (1/4 Hr)	5.7 (3/2/15) 5.2 (3/20/15)
B-10A	8+21	Welch	23.4	3' - 14': Stiff to very stiff CH 8' - 14' Very stiff to hard CL 14' - 33': Very stiff to hard CH	Dry (Drilling)	29.3 (1/28/15) 26.0 (3/2/15) 5.2 (3/20/15)

Tunneling operations and placement of storm sewer inside tunnel constructed with primary liner should comply with Sections 02426 of the latest edition of the City of Houston Standard Construction Specifications (COHSCS).

5.2.1 Loadings on Pipes

Underground utilities support the weight of the soil and water above the crown, as well as roadway traffic and any structures that exist above the utilities.

Earth Loads: For underground utilities to be installed using open cut methods, the vertical soil load W_e can be calculated as the larger of the two values from Equations (1) and (3):

$$W_e = C_d \gamma B_d^2 \qquad \text{...........Equation (1)}$$

$$C_d = [1 - e^{-2K\mu'(H/B_d)}]/(2K\mu') \quad \text{...........Equation (2)}$$

$$W_e = \gamma B_c H \quad \text{...........Equation (3)}$$

where: W_e = trench fill load, in pounds per linear foot (lb/ft);
C_d = trench load coefficient, see Plate C-2, in Appendix C;
γ = effective unit weight of soil over the conduit, in pounds per cubic foot (pcf);
B_d = trench width at top of the conduit < 1.5 B_c (ft);
B_c = outside diameter of the conduit (ft);
H = variable height of fill (ft);
when the height of fill above the top of the conduit $H_c > 2\ B_d$, $H = H_h$ (height of fill above the middle of the conduit). When $H_c < 2\ B_d$, H varies over the height of the conduit; and
$K\mu'$ = 0.1650 maximum for sand and gravel,
0.1500 maximum for saturated top soil,
0.1300 maximum for ordinary clay,
0.1100 maximum for saturated clay.

When underground conduits are located below groundwater, the total vertical dead loads should include the weight of the projected volume of water above the conduits.

Traffic Loads: The vertical stress on top of an underground conduit, p_L (psf), resulting from traffic loads (from a HS-20 truck) can be obtained from Plate C-3, in Appendix C. The live load on top of the underground conduit can be calculated from Equation (4):

$$W_L = p_L\, B_c \quad \text{...........Equation (4)}$$

where: W_L = live load on the top of the conduit (lb/ft);
p_L = vertical stress (on the top of the conduit) resulting from traffic loads (psf);
B_c = outside diameter of the conduit, (ft);

Lateral Loads: The lateral soil pressure p_l can be calculated from Equation (5); hydrostatic pressure should be added, if applicable.

$$p_l = 0.5\,(\gamma H_h + p_s) \quad \text{...........Equation (5)}$$

where: H_h = height of fill above the center of the conduit (ft);
γ = effective unit weight of soil over the conduit (pcf);
p_s = vertical pressure on conduit resulting from traffic and/or construction equipment (psf).

5.2.2 Tunnel Access Shafts

Tunnel access shafts should be constructed in accordance with Section 02400 of the latest edition of the

COHSCS. Based on Table 5, the tunnel access shafts on the south end of the W. Gray tunnel (at Boring B-8A) will encounter water bearing clayey sand, and the tunnel shaft on the north end of the W. Gray tunnel (at Boring B-8B) will encounter lean/fat clay and groundwater. The tunnel access shafts at the Welch tunnel (at Boring B-10A) will encounter lean/fat clay. Since the access shafts (especially on the west end of the tunnel) will most likely extend into water-bearing sand/silt, the access shaft walls can be supported by internally-braced, water-tight steel sheet piles.

AEC anticipates ground water control will be required for the tunnel shafts. Possible ground water control measures includes: (i) deep wells with turbine or submersible pumps; (ii) educators (for silt); (iii) water-tight sheet pile cut-off walls; or (iv) jet-grouting of sandy soils in the immediate surrounding area. Generally, the groundwater depth should be lowered at least 5 feet below the excavation bottom in accordance with Section 01578 of the latest edition of the City of Houston Standard General Requirement (COHSGR) to be able to work on a firm surface when water-bearing granular soils are encountered. If deep wells are used to dewater the excavation, extended and/or excessive dewatering can result in settlement of existing structures in the vicinity. One option to reduce the risk of settlement in these cases includes installing a series of reinjection wells around the perimeter of the construction area. General groundwater control recommendations are presented in Section 6.2 of this report. The options for dewatering presented here are for reference purposes only; it is the Contractor's responsibility to take the necessary precautions to minimize the effect on existing structures in the vicinity of the dewatering operation.

<u>Sheet Piling:</u> Design soil parameters for sheet pile design are presented on Plate C-1, in Appendix C. AEC recommends that the sheet pile design consider both short-term and long-term parameters; whichever is critical should be used for design. The determination of the pressures exerted on the sheet piles by the retained soils shall consider active earth pressure, hydrostatic pressure, and uniform surcharge (including construction equipment, soil stockpiles, and traffic load, whichever surcharge is more critical).

Sheet pile design should be based on the following considerations:

(1) Ground water elevation at the top of the ground surface on the retained side;
(2) Ground water elevation 5 feet below the bottom of the access shaft excavation (assuming dewatering operations using deep wells);
(3) Neglect cohesion for active pressure determination, see Equation (6) below;
(4) The design retained height should extend from the ground surface to the water line tunnel invert depth;

(5) A 300 psf uniform surcharge pressure from construction equipment or soil stockpiles should be considered at the top of the sheet piles; loose soil stockpiles during access shaft construction should be limited to 3 foot high or less;
(6) Use a Factor of Safety of 2.0 for passive earth pressure in front of (i.e. the shaft side) the sheet piles.

Design, construction, and monitoring of sheet piles should be performed by qualified personnel who are experienced in this operation. Sheet piles should be driven in pairs, and proper construction controls provided to maintain alignment along the wall and prevent outward leaning of the sheet piles.

Determination of Earth Pressures for Sheet Piling Design: The following method can be used for calculating earth pressure against sheet piles. Lateral pressure resulting from construction equipment, traffic loads, or other surcharge should be taken into account by adding the equivalent uniformly distributed surcharge to the design lateral pressure. Hydrostatic pressure, if any, should also be considered. The active earth pressure at depth z can be determined by Equation (6). The design soil parameters for trench bracing design are presented on Plate C-1, in Appendix C.

$$p_a = (q_s + \gamma h_1 + \gamma' h_2)K_a - 2c\sqrt{K_a} + \gamma_w h_2 \qquad \text{............Equation (6)}$$

where:
- p_a = active earth pressure (psf);
- q_s = uniform surcharge pressure (psf);
- γ, γ' = wet unit weight and buoyant unit weight of soil (pcf);
- h_1 = depth from ground surface to groundwater table (ft);
- h_2 = z-h_1, depth from groundwater table to the point under consideration (ft);
- z = depth below ground surface for the point under consideration (ft);
- K_a = coefficient of active earth pressure;
- c = cohesion of clayey soils (psf); c can be omitted conservatively;
- γ_w = unit weight of water, 62.4 pcf.

Pressure distribution for the practical design of struts in open cuts for clays and sands are illustrated on Plates D-1 through D-3, in Appendix D.

Bottom Stability: In open-cuts, it is necessary to consider the possibility of the bottom failing by heaving, due to the removal of the weight of excavated soil. Heaving typically occurs in soft plastic clays when the excavation depth is sufficiently deep enough to cause the surrounding soil to displace vertically due to bearing capacity failure of the soil beneath the excavation bottom, with a corresponding upward movement of the soils in the bottom of the excavation. In fat and lean clays, heave normally does not occur unless the ratio of Critical Height to Depth of Cut approaches one. In very sandy and silty lean clays and granular soils, heave can occur if an artificially large head of water is created due to installation of impervious sheeting while bracing the cut. This can be mitigated if groundwater is lowered below the excavation by dewatering the area. Guidelines for evaluating bottom stability in clay soils are presented on Plate D-4, in Appendix D.

If the excavation extends below groundwater, and the soils at or near the bottom of the excavation are mainly sands or silts, the bottom can fail by blow-out (boiling) when a sufficient hydraulic head exists. The potential for boiling or in-flow of granular soils increases where the groundwater is pressurized. To reduce the potential for boiling of excavations terminating in granular soils below pressurized groundwater, the groundwater table should be lowered at least 5 feet below the excavation in accordance with Section 01578 of the latest edition of the COHSGR.

Calcareous nodules, silt/sand seams, and fat clays with slickensides were encountered in some of the borings. These secondary structures may become sources of localized instability when they are exposed during excavation, especially when they become saturated. Such soils have a tendency to slough or cave in when not laterally confined, such as in trench excavations. The Contractor should be aware of the potential for cave-in of the soils. Low plasticity soils (silts and clayey silts) will lose strength and may behave like granular soils when saturated.

Reaction Walls: Reaction walls (if used) will be part of the tunnel shaft walls; they will be rigid structures and support tunneling operations by mobilizing passive pressures of the soils behind the walls. The passive earth pressure can be calculated using Equation (8). A factor of safety of 2.0 should be used for passive earth pressure design. The design soil parameters are presented on Plate C-1, in Appendix C.

$$p_p = \gamma z K_p + 2c(K_p)^{1/2} \qquad \text{...........Equation (8)}$$

where,
- p_p = passive earth pressure (psf);
- γ = wet unit weight of soil (pcf);
- z = depth below ground surface for the point under consideration (ft);
- K_p = coefficient of passive earth pressure;
- c = cohesion of clayey soils (psf).

Due to subsurface variations, soils with different strengths and characteristics will likely be encountered at a given location. The soil resulting in the lowest passive pressure should be used for design of the walls. The soil conditions should be checked by geotechnical personnel to confirm the recommended soil parameters.

5.2.3 Tunnel Face Stability during Construction

5.2.3.1 General

The stability of a tunnel face is governed primarily by ground water and subsurface soil conditions, type of tunnel machine used, and workmanship. Based on the subsurface conditions encountered in our borings and the proposed invert depths (see Table 5 in Section 5.2 of this report), we anticipate that stiff to hard fat clay and clayey sand will be encountered within the tunnel zone of the W. Gray tunnel (Borings B-8A and B-8B), and that very stiff to hard lean/fat clay will be encountered within the tunnel zone of the Welch tunnel (Boring B-10A). Secondary features such as sand or silt partings/seams/pockets/layers were also encountered within the cohesive soils, and could be significant at some locations. In addition, the type and property of subsurface soils are subject to change between borings, and may be different at locations away from our borings.

When granular soils are encountered during construction the tunnel face can become unstable. Granular soils below ground water will tend to flow into the excavation hole; granular soils above the ground water level will generally not stand unsupported but will tend to ravel until a stable slope is formed at the face with a slope equal to the angle of repose of the material in a loose state. Thus, granular soils are generally considered unstable in an unsupported excavation face; uncontrolled flowing soil can result in large loss of ground.

5.2.3.2 Anticipated Ground Behavior

A Stability Factor, $N_t = (P_z - P_a)/C_u$ may be used to evaluate the stability of an unsupported bore face in cohesive soils (N_t is not applicable to granular soils), where P_z is the overburden pressure to the bore centerline; P_a is the equivalent uniform interior pressure applied to the face; and C_u is the soil undrained shear strength. For bore/auger operations, no interior pressure is applied. Generally, N_t values of 4 or less are desirable as it represents a practical limit below which tunneling may be accomplished without significant difficulty. Higher N_t values usually lead to large deformations of the soil around the bore and problems associated with increased subsidence. It should be noted that the exposure time of the face is most important; with time, creep of the soil will occur, resulting in a reduction of shear strength. The N_t values will therefore increase when construction is slow.

Where granular or soft cohesive soils are encountered, the Contractor should make provisions to stabilize the tunnel excavations. The Contractor should not base their bid on the above information alone, since granular soils may be encountered between boring locations; the Contractor should verify the subsurface conditions between boring locations or add a contingency.

We also estimated the maximum settlements [caused by volume loss if a slurry face machine (SFM) or earth pressure balance tunnel boring machine (EPB) is NOT used] at the proposed tunnel location and the results are included in Table 6.

Table 6. Tunnel Face Stability Factor and Estimated Settlements along Tunnel Alignment

Soil Boring/ Station	Tunnel Segment	Tunnel Invert Depth (ft)	Anticipated Soil Types in Tunnel Zone	Stability Factor N_t	S_{max} (in)	Note/Suggestion
B-8A/ 19+09	W. Gray	24.9	Very stiff to hard Lean Clay (CL) Very stiff Fat Clay (CH) Water-bearing Clayey Sand (SC)	3.2	0.75	Mixed ground conditions under water, suggest using SFM or EPB TBM
B-8B/ 20+60	W. Gray	25.1	Stiff to hard Fat Clay (CH), saturated Stiff to hard Lean Clay (CL), saturated	1.3	0.25	Potential swelling ground due to very high plasticity CH
B-10A/ 8+21	Welch	23.4	Stiff to hard Fat Clay (CH)	1.7	0.25	Potential swelling ground due to very high plasticity CH

Note: S_{max} = Estimated settlement along the tunnel alignment due to volume loss if slurry face machine (SFM) or EPB are not used; not including consolidation settlement.

Based on Table 6, it should be noted that the estimated settlement at Boring B-8A's location is approximately 0.75 inches (which does not include consolidation settlement) or more, and dewatering at Boring B-8A's location will also cause additional settlement due to increases in effective stress of the soil strata. The information in this report should be reviewed so that appropriate tunneling equipment and operation can be planned and factored into the construction plan and cost estimate. If the estimated settlement is too high, we suggest that the tunnel construction consider the use of: (i) a SFM or EPB TBM; (ii) jet grout to stabilize the saturated granular soils; or (iii) micro-tunneling. The choice of tunneling machine should be selected by the Contractor. Plate D-5 in Appendix D provides a general guideline for TBM selection. Tunnel construction should be in accordance with Section 02426 of the latest edition of the COHSCS.

5.2.3.3 Influence of Tunneling on Existing Structures

Ground Subsidence: Tunneling in soft ground often induces some degree of settlement (ground subsidence) of the overlying ground surface. If such settlement is excessive, it may cause damage to existing structures and services located above and/or near the tunnel zone.

The tunnel influence zone is assumed to extend a distance of about 2.5i from the center of the auger tunnel, as shown on Plate D-6, in Appendix D. We estimated the resulting influence zones (extending from the centerline of the tunnel) to range from approximately 19 to 25 feet at Borings B-8A and B-8B for the W. Gray tunnel and approximately 25 feet at Boring B-10A for the Welch tunnel; although the values of tunnel influence zone presented are rough estimates. The estimated maximum settlements [caused by volume loss if a TBM is not used] along the tunnel alignment at the proposed tunnel locations are included in Table 6 of this report.

AEC emphasizes that the size of the influence zone of a tunnel is difficult to determine because several factors influence the response of the soil to tunneling operations including type of soil, ground water level and control method, type of tunneling equipment, tunneling operations, experience of operator, and other construction in the vicinity. Methods to prevent movement and/or distress to existing structures will require the services of a specialty contractor.

5.2.4 Measures to Reduce Distress from Tunneling

To control tunneling face loss and reduce potential impact on existing foundations and structures, AEC recommends the use of a steel casing (or equivalent methods) to support the tunnel excavation during tunnel construction. Considering the ground conditions discussed in Table 6 of this report, AEC recommends that the following tunneling operations be considered: (i) use a pressurized slurry TBM and keep the pressure at least equal to if not greater than the combined soil and groundwater pressure in the ground at the tunnel level; and (ii) if excessive voids occur during tunneling, the contractor should immediately and completely grout the annular space between the steel casing and the ground at the tail of the machine, in accordance with Section 02431 of the latest edition of the COHSCS. It should be noted that grouting may increase friction resistance while advancing the casing and the contractor will need to address this condition as part of his tunnel work plan. Plate D-7, in Appendix D provides a general guideline for selection of grouting material. The tunneling machine selection, tunneling operation, and grouting (as necessary) will be the full responsibility of the Contractor.

To reduce the potential for the tunneling to influence existing foundations or structures, we recommend that the outer edge of the influence zone of the tunnel be a minimum of 5 feet from the outer edge of the bearing (stress) zone of existing foundations. The bearing (stress) zone is defined by a line drawn downward from the outer edge of an existing foundation and inclined at an angle of 45 degrees to the vertical.

We recommend that the following situations be evaluated on a case by case basis, where:

- tunneling cannot be located farther than the minimum distance recommended above;
- tunneling cannot be located outside the stress zone of the foundations for existing structures;
- unstable soils are encountered near existing structures;
- heavily loaded or critical structures are located close to the influence zone of the tunnels;

As an option, existing structure foundations should be protected by adequate shoring or strengthened by underpinning or other techniques, provided that tunneling cannot be located outside the stress zone of the existing foundations.

Disturbance and loss of ground from the tunneling operation may create surface soil disturbance and subsidence which in turn may cause distress to existing structures (including underground utilities and pavements) located in the zone of soil disturbance. Any open-cut excavation in the proposed tunneling areas should be adequately shored.

5.2.5 Monitoring Existing Structures

The Contractor should be responsible for monitoring existing structures nearby and taking necessary action to mitigate impact to adjacent structures. Existing structures located close to the proposed construction excavations should be surveyed prior to construction and pre-existing conditions of such structures and their vicinity be adequately recorded. This can be accomplished by conducting a pre-construction survey, taking photographs and/or video, and documenting existing elevations, cracks, settlements, and other existing distress in the structures. The monitoring should include establishment of elevation monitor stations, crack gauges, and inclinometers, as required. The monitoring should be performed before, periodically during, and after construction. The data should be reviewed by qualified engineers in a timely manner to evaluate the impact on existing structures and develop plans to mitigate the impact, should it be necessary.

6.0 CONSTRUCTION CONSIDERATIONS

6.1 Site Preparation

To mitigate site problems that may develop following prolonged periods of rainfall, it is essential to have adequate drainage to maintain a relatively dry and firm surface prior to starting any work at the site. Adequate drainage should be maintained throughout the construction period. Methods for controlling surface runoff and ponding include proper site grading, berm construction around exposed areas, and installation of sump pits with pumps.

6.2 Groundwater Control

The need for groundwater control will depend on the depth of excavation relative to the groundwater depth at the time of construction. In the event that there is heavy rain prior to or during construction, the groundwater table may be higher than indicated in this report; higher seepage is also likely and may require a more extensive groundwater control program. In addition, groundwater may be pressurized in certain areas of the alignment, requiring further evaluation and consideration of the excess hydrostatic pressures. Groundwater control should be in general accordance with Section 01578 of the latest edition of the COHSGR.

The Contractor should be responsible for selecting, designing, constructing, maintaining, and monitoring a groundwater control system and adapt his operations to ensure the stability of the excavations. Groundwater information presented in Section 4.1 and elsewhere in this report, along with consideration for potential environmental and site variation between the time of our field exploration and construction, should be incorporated in evaluating groundwater depths. The following recommendations are intended to guide the Contractor during design and construction of the dewatering system.

In cohesive soils seepage rates are lower than in granular soils and groundwater is usually collected in sumps and channeled by gravity flow to storm sewers. If cohesive soils contain significant secondary features, seepage rates will be higher. This may require larger sumps and drainage channels, or if significant granular layers are interbedded within the cohesive soils, methods used for granular soils may be required. Where it is present, pressurized groundwater will also yield higher seepage rates.

Groundwater for excavations within saturated sands can be controlled by the installation of wellpoints. The practical maximum dewatering depth for well points is about 15 feet. When groundwater control is required below 15 feet, possible ground water control measures include: (i) deep wells with turbine or submersible pumps; (ii) multi-staged well points; or (iii) water-tight sheet pile cut-off walls. Generally, the groundwater depth should be lowered at least 5 feet below the excavation bottom (in accordance with Section 01578 of the latest edition of the COHSGR) to be able to work on a firm surface when water-bearing granular soils are encountered.

Extended and/or excessive dewatering can result in settlement of existing structures in the vicinity; the Contractor should take the necessary precautions to minimize the effect on existing structures in the vicinity of the dewatering operation. We recommend that the Contractor verify the groundwater depths and seepage rates prior to and during construction and retain the services of a dewatering expert (if necessary) to assist him in identifying, implementing, and monitoring the most suitable and cost-effective method of controlling groundwater.

Note that extended and/or excessive dewatering can result in differential settlement of existing adjacent structures as the groundwater table is lowered. Special care should be exercised to prevent a change of the groundwater level below structures when performing dewatering operations for the storm sewer installation. One option to reduce such risk includes using a sheet pile cutoff wall to minimize seepage into the excavation, combined with a series of monitoring and reinjection wells (to maintain the ground table) around the construction area.

For open cut construction in cohesive soils, the possibility of bottom heave must be considered due to the removal of the weight of excavated soil. In lean and fat clays, heave normally does not occur unless the ratio of Critical Height to Depth of Cut approaches one. In silty clays, heave does not typically occur unless an artificially large head of water is created through the use of impervious sheeting in bracing the cut. Guidelines for evaluating bottom stability are presented in Section 5.2.2 of this report.

Sheet Piling: Recommendations for sheet pile design are presented in Section 5.2.2 of this report. Design, construction, and monitoring of sheet piles should be performed by qualified personnel who are experienced in this operation. Sheet piles should be driven in pairs, and proper construction controls provided to maintain alignment along the wall and prevent outward leaning of the sheet piles. Construction of the sheet piles should be in accordance with the latest edition of the COHSCS, or equivalent standard, such as Item 407 of the 2004 TxDOT Standard Specifications.

6.3 Construction Monitoring

Pavement construction and subgrade preparation, as well as excavation, bedding, and backfilling of underground utilities should be monitored by qualified geotechnical professionals to check for compliance with project documents and changed conditions, if encountered. AEC should be allowed to review the design and construction plans and specifications prior to release to check that the geotechnical recommendations and design criteria presented herein are properly interpreted.

6.4 Monitoring of Existing Structures

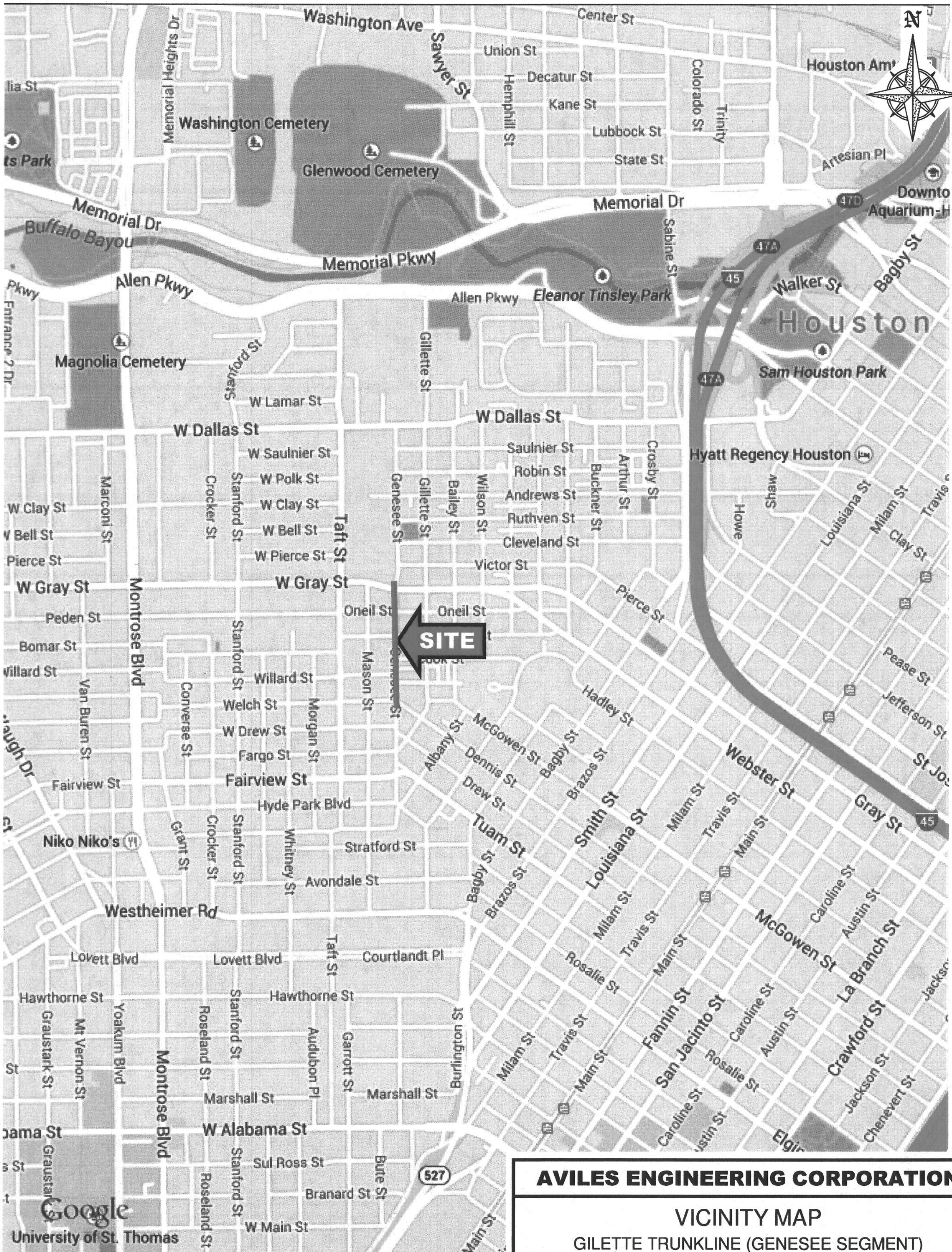
Existing structures in the vicinity of the proposed alignment should be closely monitored prior to, during, and for a period after excavation. Several factors (including soil type and stratification, construction methods, weather conditions, other construction in the vicinity, construction personnel experience and supervision) may impact ground movement in the vicinity of the alignment. We therefore recommend that the Contractor be required to survey and adequately document the condition of existing structures in the vicinity of the proposed alignments.

7.0 LIMITATIONS

The information contained in this report summarizes conditions found on the dates the borings were drilled. The attached boring logs are true representations of the soils encountered at the specific boring locations on the dates of drilling. Reasonable variations from the subsurface information presented in this report should be anticipated. If conditions encountered during construction are significantly different from those presented in this report; AEC should be notified immediately.

This investigation was performed using the standard level of care and diligence normally practiced by recognized geotechnical engineering firms in this area, presently performing similar services under similar circumstances. This report is intended to be used in its entirety. The report has been prepared exclusively for the project and location described in this report. If pertinent project details change or otherwise differ from those described herein, AEC should be notified immediately and retained to evaluate the effect of the changes on the recommendations presented in this report, and revise the recommendations if necessary. The recommendations presented in this report should not be used for other structures located along these alignments or similar structures located elsewhere, without additional evaluation and/or investigation.

APPENDIX A



AVILES ENGINEERING CORPORATION

VICINITY MAP

GILETTE TRUNKLINE (GENESEE SEGMENT)
WBS NO. M-410290-0003-4
HOUSTON, TEXAS

AEC PROJECT NO.: G166B-12	DATE: 03-02-15	SOURCE DRAWING PROVIDED BY: GOOGLE MAPS
APPROX. SCALE: N.T.S.	DRAFTED BY: WLW	PLATE NO.: PLATE A-1

AVILES ENGINEERING CORPORATION

BORING LOCATION PLAN
GILLETTE TRUNKLINE (GENESEE SEGMENT)
WBS NO. M-410290-0003-3
HOUSTON, TEXAS

AEC PROJECT NO.:	DATE:	SOURCE DRAWING PROVIDED BY:
G166-12B	03-17-15	GOOGLE EARTH PRO
APPROX. SCALE:	DRAFTED BY:	PLATE NO.:
1" = 100'	BpJ	PLATE A-2

AVILES ENGINEERING CORP.
GEOTECHNICAL ENGINEERS

PROJECT: Gilette Trunkline (Genesee Segment) BORING B-8A

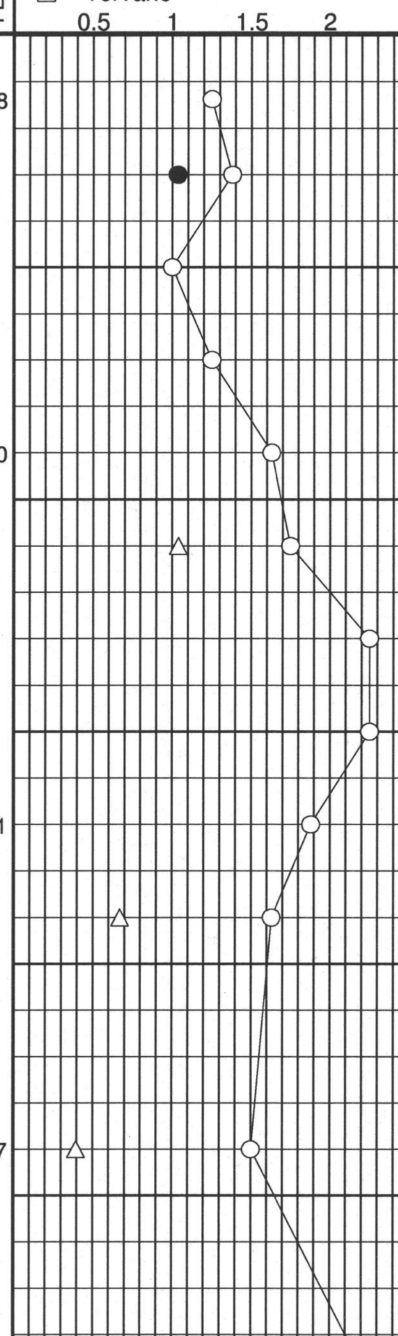
COH WBS No. M-410290-0003-3 TYPE 4" Dry Auger DATE 1/27/15

DESCRIPTION
Survey Coordinates (TSPC, Surface):
Easting: 3116085.34
Northing: 13840879.89
Elevation: 49.8

ELEVATION IN FEET	DEPTH IN FEET	DESCRIPTION	S.P.T. BLOWS / FT.	-200 MESH	DRY DENSITY, PCF	MOISTURE CONTENT, %	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
	0	Pavement: 2" asphalt							
		Base: 7" sand, shell, and gravel							
48		Very stiff, gray and olive Fat Clay w/Sand (CH)		84		21	58	20	38
					106	21			
44	4	Stiff to very stiff, tan and gray Fat Clay (CH), with slickensides -with calcareous nodules 4'-10'				23			
						23			
	8	-red, tan, and gray 8'-10'		99		32	78	28	50
40		Very stiff to hard, gray and dark green Lean Clay (CL), with abundant silt partings			115	18			
	12	-with silty sand partings 12'-14'				16			
36		-gray and red, with fat clay seams 14'-16'				21			
32	16	Very stiff, light gray Fat Clay (CH), with silty sand seams and sandy clay pockets		93		22	60	19	41
		Light gray and red Clayey Sand (SC) -with silty clay seams 18'-20'			100	26			
28	20	-with abundant silt partings 23'-25'							
24	24			45	104	22	36	19	17
	28								

SHEAR STRENGTH, TSF

△ Confined Compression
● Unconfined Compression
○ Pocket Penetrometer
□ Torvane



BORING DRILLED TO 40 FEET WITHOUT DRILLING FLUID
WATER ENCOUNTERED AT 28 FEET WHILE DRILLING
WATER LEVEL AT 23.2 FEET AFTER 1/4 HR
DRILLED BY V&S DRAFTED BY CHL LOGGED BY BPJ

PROJECT NO. G166-12 Addendum No. 8 PLATE A-3

AVILES ENGINEERING CORP.
GEOTECHNICAL ENGINEERS

PROJECT: Gilette Trunkline (Genesee Segment)

BORING B-8A

COH WBS No. M-410290-0003-3

TYPE 4" Dry Auger

DATE 1/27/15

ELEVEVATION IN FEET	DEPTH IN FEET	SYMBOL	SAMPLE INTERVAL	DESCRIPTION	S.P.T. BLOWS / FT.	-200 MESH	DRY DENSITY, PCF	MOISTURE CONTENT, %	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
				Hard, gray, red, and tan Fat Clay (CH), with slickensides		99		30	73	28	45
20											
	32										
				-light gray 33'-40'							
16								15			
	36										
12								17			
	40			Termination depth = 40 feet.							
8											
	44										
4											
	48										
0											
	52										
-4											
	56										

SHEAR STRENGTH, TSF

△ Confined Compression
● Unconfined Compression
○ Pocket Penetrometer
□ Torvane

0.5 1 1.5 2

BORING DRILLED TO 40 FEET WITHOUT DRILLING FLUID
WATER ENCOUNTERED AT 28 FEET WHILE DRILLING
WATER LEVEL AT 23.2 FEET AFTER 1/4 HR
DRILLED BY V&S DRAFTED BY CHL LOGGED BY BPJ

AVILES ENGINEERING CORP.
GEOTECHNICAL ENGINEERS

PROJECT: Gilette Trunkline (Genesee Segment) BORING B-8B

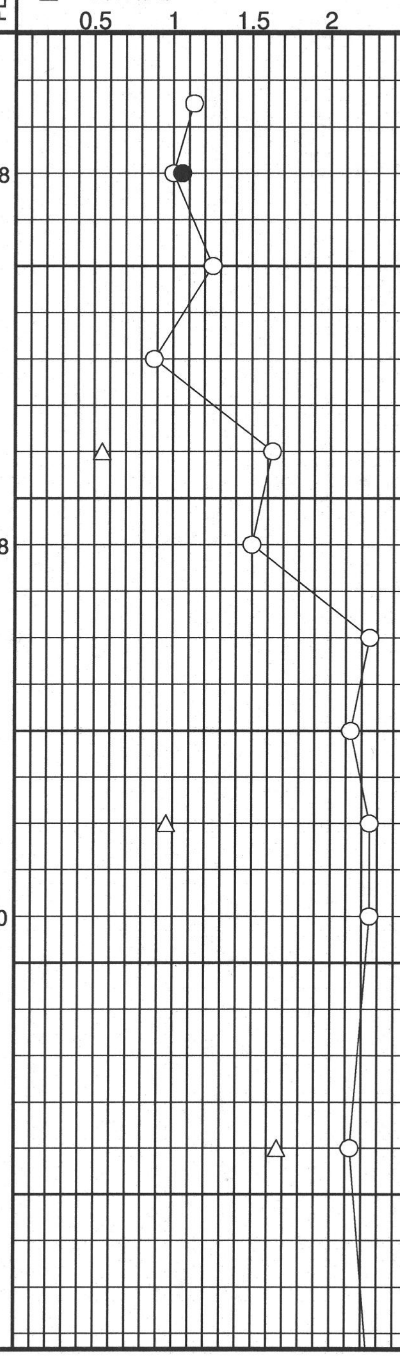
COH WBS No. M-410290-0003-3 TYPE 4" Dry Auger/Wet Rotary DATE 1/28/15

DESCRIPTION
Survey Coordinates (TSPC, Surface):
Easting: 3116068.37
Northing: 13841030.43
Elevation: 50.1

DESCRIPTION	S.P.T. BLOWS / FT.	-200 MESH	DRY DENSITY, PCF	MOISTURE CONTENT, %	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
Pavement: 8.5" concrete							
Base: 3.5" cement stabilized sand and gravel				27			
Fill: very stiff, dark gray Fat Clay (CH), with lime stabilized clay seams and organics		87	106	22	57	19	38
Stiff to very stiff, tan Fat Clay (CH), with ferrous and calcareous nodules				24			
				33			
			99	26			
Very stiff to hard, gray, red, and tan Sandy Lean Clay (CL), with calcareous nodules		70		18	41	13	28
-with fat clay seams 12'-14'				15			
Stiff to hard, gray Fat Clay (CH), with slickensides -with sandy clay seams 14'-16'				21			
-light gray and red, with silty clay seams 16'-18'			106	21			
-red and brown 18'-20' -boring cave in at 19.2' during drilling		100		24	65	25	40
Stiff to hard, gray and tan Lean Clay (CL), with slickensides and fat clay seams			117	16			

SHEAR STRENGTH, TSF

△ Confined Compression
● Unconfined Compression
○ Pocket Penetrometer
□ Torvane



ELEVATION IN FEET: 48, 44, 40, 36, 32, 28, 24
DEPTH IN FEET: 0, 4, 8, 12, 16, 20, 24, 28

BORING DRILLED TO 20 FEET WITHOUT DRILLING FLUID
WATER ENCOUNTERED AT 20 FEET WHILE DRILLING
WATER LEVEL AT 17.6 FEET AFTER 1/4 HR
DRILLED BY V&S DRAFTED BY CHL LOGGED BY MRB

PROJECT NO. G166-12

Addendum No. 8

PLATE A-4

AVILES ENGINEERING CORP. GEOTECHNICAL ENGINEERS

PROJECT: **Gilette Trunkline (Genesee Segment)**

BORING **B-8B**

COH WBS No. **M-410290-0003-3**

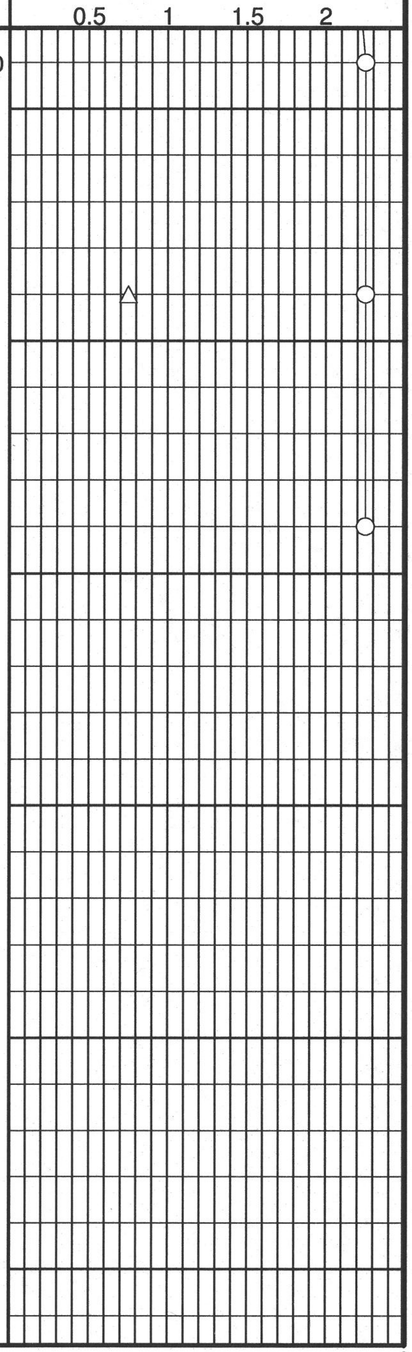
TYPE **4" Dry Auger/Wet Rotary**

DATE **1/28/15**

ELEVATION IN FEET	DEPTH IN FEET	SYMBOL	SAMPLE INTERVAL	DESCRIPTION	S.P.T. BLOWS / FT.	-200 MESH	DRY DENSITY, PCF	MOISTURE CONTENT, %	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
				Stiff to hard, Lean Clay (Cont...) -with calcareous nodules 28'-40'		85		16	46	16	30
20											
	32										
16							106	22			
	36										
12								21			
	40			Termination depth = 40 feet.							
8											
	44										
4											
	48										
0											
	52										
-4											
	56										

SHEAR STRENGTH, TSF

△ Confined Compression
● Unconfined Compression
○ Pocket Penetrometer
□ Torvane



BORING DRILLED TO **20** FEET WITHOUT DRILLING FLUID
WATER ENCOUNTERED AT **20** FEET WHILE DRILLING
WATER LEVEL AT **17.6** FEET AFTER **1/4 HR**
DRILLED BY **V&S** DRAFTED BY **CHL** LOGGED BY **MRB**

PROJECT NO. G166-12

Addendum No. 8

PLATE A-4

AVILES ENGINEERING CORP. GEOTECHNICAL ENGINEERS

PROJECT: Gilette Trunkline (Genesee Segment) BORING B-10A

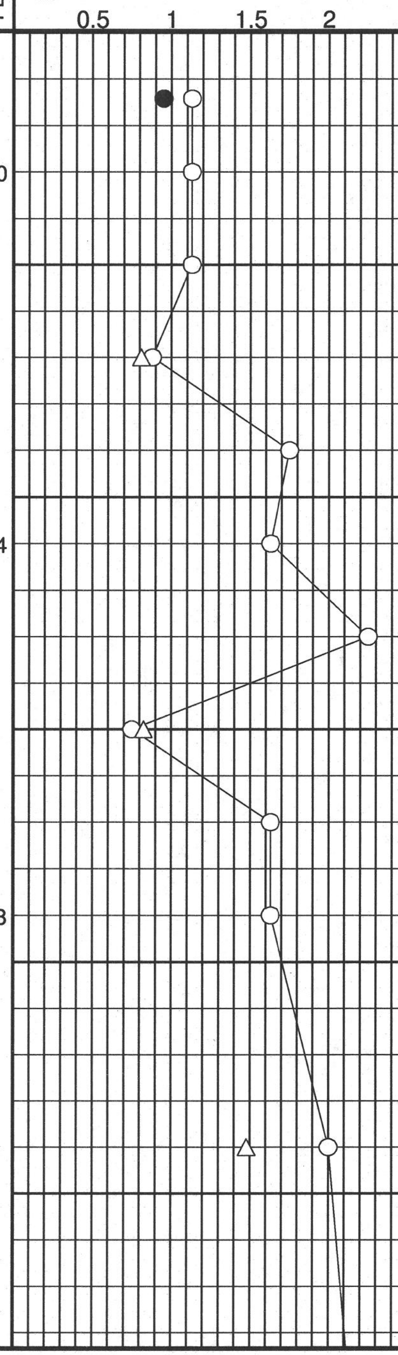
COH WBS No. M-410290-0003-3 TYPE 4" Dry Auger DATE 1/27/15

DESCRIPTION	S.P.T. BLOWS / FT.	-200 MESH	DRY DENSITY, PCF	MOISTURE CONTENT, %	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
Survey Coordinates (TSPC, Surface): Easting: 3116111.42 Northing: 13839792.91 Elevation: 49.2							
Pavement: 4" asphalt							
Base: 6" sand and gravel							
Stiff to very stiff, dark brown Fat Clay (CH)			95	27			
-with ferrous nodules 2'-4'		93		26	72	22	50
-olive gray and dark brown 4'-6'				29			
-tan and light gray, with ferrous nodules 6'-8'			92	30			
Very stiff to hard, gray Lean Clay w/Sand (CL) -with fat clay pockets 8'-10'				15			
-olive and tan, with silty sand seams 10'-14'		78		18	38	14	24
				15			
Stiff to very stiff, gray and olive Fat Clay w/ Sand (CH), with slickensides -with clayeys sand seams 14'-16'			104	27			
-with clayey sand pockets 16'-18'				16			
		79		16	75	27	48
Very stiff to hard, red, brown and light gray Fat Clay (CH), with slickensides			87	34			

Elevation in feet: 48, 44, 40, 36, 32, 28, 24. Depth in feet: 0, 4, 8, 12, 16, 20, 24, 28.

SHEAR STRENGTH, TSF

△ Confined Compression
● Unconfined Compression
○ Pocket Penetrometer
□ Torvane



BORING DRILLED TO 40 FEET WITHOUT DRILLING FLUID
WATER ENCOUNTERED AT N/A FEET WHILE DRILLING
WATER LEVEL AT 25.7 FEET AFTER 24 HRS
DRILLED BY V&S DRAFTED BY CHL LOGGED BY BPJ

PROJECT NO. G166-12

Addendum No. 8

PLATE A-5

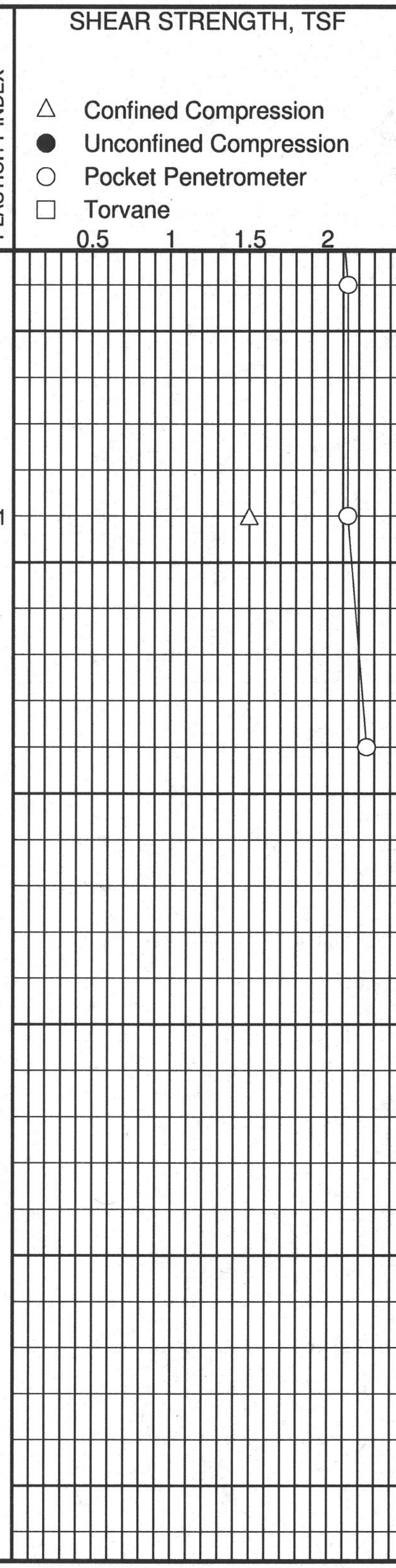
PROJECT: Gilette Trunkline (Genesee Segment)

AVILES ENGINEERING CORP. GEOTECHNICAL ENGINEERS

BORING B-10A

COH WBS No. M-410290-0003-3 TYPE 4" Dry Auger DATE 1/27/15

ELEVATION IN FEET	DEPTH IN FEET	DESCRIPTION	S.P.T. BLOWS / FT.	-200 MESH	DRY DENSITY, PCF	MOISTURE CONTENT, %	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
20		Very stiff to hard, Fat Clay (Cont...) -with calcareous nodules 28'-30'				27			
	32								
16				95	87	36	86	35	51
	36								
12		Hard, tan and light gray Lean Clay (CL), with silt partings				14			
	40	Termination depth = 40 feet.							
8									
	44								
4									
	48								
0									
	52								
-4									
	56								



BORING DRILLED TO 40 FEET WITHOUT DRILLING FLUID

WATER ENCOUNTERED AT N/A FEET WHILE DRILLING

WATER LEVEL AT 25.7 FEET AFTER 24 HRS

DRILLED BY V&S DRAFTED BY CHL LOGGED BY BPJ

PROJECT NO. G166-12

PLATE A-5

KEY TO SYMBOLS

Symbol Description

Strata symbols

Symbol	Description
	Paving
	Fill
	High plasticity clay
	Low plasticity clay
	Clayey sand

Misc. Symbols

Symbol	Description
	Water table depth during drilling
	Subsequent water table depth
○	Pocket Penetrometer
●	Unconfined Compression
△	Confined Compression

Soil Samplers

Symbol	Description
	Auger
	Undisturbed thin wall Shelby tube
	Rock core

PLATE A-6

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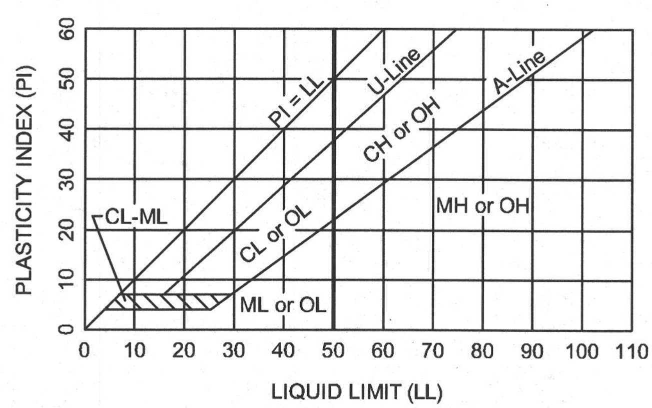
CLASSIFICATION OF SOILS FOR ENGINEERING PURPOSES

ASTM Designation D-2487

MAJOR DIVISIONS				GROUP SYMBOL	TYPICAL NAMES
COARSE-GRAINED SOILS (Less than 50% passes No. 200 sieve)	GRAVELS (Less than 50% of coarse fraction passes No. 4 sieve)	CLEAN GRAVELS (Less than 5% passes No. 200 sieve)		GW	Well-graded gravel, well-graded gravel with sand
				GP	Poorly-graded gravel, poorly-graded gravel with sand
		GRAVELS WITH FINES (More than 12% passes No. 200 sieve)	Limits plot below "A" line & hatched zone on plasticity chart	GM	Silty gravel, silty gravel with sand
			Limits plot above "A" line & hatched zone on plasticity chart	GC	Clayey gravel, clayey gravel with sand
	SANDS (50% or more of coarse fraction passes No. 4 sieve)	CLEAN SANDS (Less than 5% passes No. 200 sieve)		SW	Well-graded sand, well-graded sand with gravel
				SP	Poorly-graded sand, poorly-graded sand with gravel
		SANDS WITH FINES (More than 12% passes No. 200 sieve)	Limits plot below "A" line & hatched zone on plasticity chart	SM	Silty sand, silty sand with gravel
			Limits plot above "A" line & hatched zone on plasticity chart	SC	Clayey sand, clayey sand with gravel
FINE-GRAINED SOILS (50% or more passes No. 200 sieve)	SILTS AND CLAYS (Liquid Limit Less Than 50%)			ML	Silt, silt with sand, silt with gravel, sandy silt, gravelly silt
				CL	Lean clay, lean clay with sand, lean clay with gravel, sandy lean clay, gravelly lean clay
				OL	Organic clay, organic clay with sand, sandy organic clay, organic silt, sandy organic silt
	SILTS AND CLAYS (Liquid Limit 50% or More)			MH	Elastic silt, elastic silt with sand, sandy elastic silt, gravelly elastic silt
				CH	Fat clay, fat clay with sand, fat clay with gravel, sandy fat clay, gravelly fat clay
				OH	Organic clay, organic clay with sand, sandy organic clay, organic silt, sandy organic silt

NOTE: Coarse soils between 5% and 12% passing the No. 200 sieve and fine-grained soils with limits plotting in the hatched zone of the plasticity chart are to have dual symbols.

PLASTICITY CHART



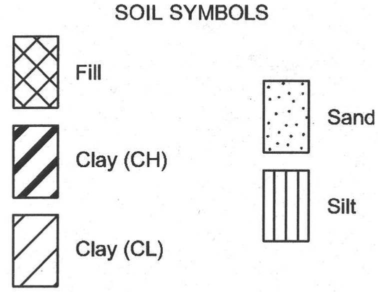
Equation of A-Line: Horizontal at PI=4 to LL=25.5, then PI=0.73(LL-20)

Equation of U-Line: Vertical at LL=16 to PI=7, then PI=0.9(LL-8)

DEGREE OF PLASTICITY OF COHESIVE SOILS

Degree of Plasticity	Plasticity Index
None	0 - 4
Slight	5 - 10
Medium	11 - 20
High	21 - 40
Very High	>40

SOIL SYMBOLS



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TERMS USED ON BORING LOGS

SOIL GRAIN SIZE

U.S. STANDARD SIEVE

	6"	3"	3/4"	#4	#10	#40	#200	
BOULDERS	COBBLES	GRAVEL		SAND			SILT	CLAY
		COARSE	FINE	COARSE	MEDIUM	FINE		
152	76.2	19.1	4.76	2.00	0.420	0.074	0.002	

SOIL GRAIN SIZE IN MILLIMETERS

STRENGTH OF COHESIVE SOILS

Consistency	Undrained Shear Strength, Kips per Sq. ft.
Very Soft	less than 0.25
Soft	0.25 to 0.50
Firm	0.50 to 1.00
Stiff	1.00 to 2.00
Very Stiff	2.00 to 4.00
Hard	greater than 4.00

RELATIVE DENSITY OF COHESIONLESS SOILS FROM STANDARD PENETRATION TEST

Very Loose	<4 bpf
Loose	5-10 bpf
Medium Dense	11-30 bpf
Dense	31-50 bpf
Very Dense	>50 bpf

SPLIT-BARREL SAMPLER DRIVING RECORD

Blows per Foot	Description
25	25 blows driving sampler 12 inches, after initial 6 inches of seating.
50/7"	50 blows driving sampler 7 inches, after initial 6 inches of seating.
Ref/3"	50 blows driving sampler 3 inches, during initial 6-inches seating interval.

NOTE: To avoid change to sampling tools, driving is limited to 50 blows during or after seating interval.

DRY STRENGTH ASTM D2488

None	Dry specimen crumbles into powder with mere pressure of handling
Low	Dry specimen crumbles into powder with some finger pressure
Medium	Dry specimen breaks into pieces or crumbles with considerable pressure
High	Dry specimen cannot be broken with finger pressure, it can be broken between thumb and hard surface
Very High	Dry specimen cannot be broken between thumb and hard surface

MOISTURE CONDITION ASTM D2488

Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water

SOIL STRUCTURE

Slickensided	Having planes of weakness that appear slick and glossy. The degree of slickensidedness depends upon the spacing of slickensides and the easiness of breaking along these planes.
Fissured	Containing shrinkage or relief cracks, often filled with fine sand or silt; usually more or less vertical.
Pocket	Inclusion of material of different texture that is smaller than the diameter of the sample.
Parting	Inclusion less than 1/8 inch thick extending through the sample.
Seam	Inclusion 1/8 inch to 3 inches thick extending through the sample.
Layer	Inclusion greater than 3 inches thick extending through the sample.
Laminated	Soil sample composed of alternating partings or seams of different soil types.
Interlayered	Soil sample composed of alternating layers of different soil types.
Intermixed	Soil sample composed of pockets of different soil types and layered or laminated structure is not evident.
Calcareous	Having appreciable quantities of calcium material.

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ASTM & TXDOT DESIGNATION FOR SOIL LABORATORY TESTS

NAME OF TEST	ASTM TEST DESIGNATION	TXDOT TEST DESIGNATION
Moisture Content	D 2216	Tex-103-E
Specific Gravity	D 854	Tex-108-E
Sieve Analysis	D 421 D 422	Tex-110-E (Part 1)
Hydrometer Analysis	D 422	Tex-110-E (Part 2)
Minus No. 200 Sieve	D 1140	Tex-111-E
Liquid Limit	D 4318	Tex-104-E
Plastic Limit	D 4318	Tex-105-E
Shrinkage Limit	D 427	Tex-107-E
Standard Proctor Compaction	D 698	Tex-114-E
Modified Proctor Compaction	D 1557	Tex-113-E
Permeability (constant head)	D 2434	-
Consolidation	D 2435	-
Direct Shear	D 3080	-
Unconfined Compression	D 2166	-
Unconsolidated-Undrained Triaxial	D 2850	Tex-118-E
Consolidated-Undrained Triaxial	D 4767	Tex-131-E
Pinhole Test	D 4647	-
California Bearing Ratio	D 1883	-
Unified Soil Classification System	D 2487	Tex-142-E

SUMMARY OF TEST RESULTS	Project Name: GILETTE TRUNKLINE (GENESEE SEGMENT) PAVING AND DRAINAGE															
	WBS Number: M-410290-0003-3															
Aviles Engineering Corporation	AEC Project Number: G166-12B															
Boring No.	Sample				SPT (blows/ft)	Water Content (%)	Dry Density (pcf)	Atterberg Limits			Percent Passing Sieve #200 (%)	Shear Strength (tsf)				Type of Material
	No.	Depth (ft)		Type				LL (%)	PL (%)	PI (%)		Unconfined Compression	UU (confining pressure, psi)	Torvane	Pocket Penetrometer	
		Top	Bottom													
B-8A	1	0.0	2.0	UD		21		58	20	38	83.7				2.50	Fat Clay w/Sand (CH)
	2	2.0	4.0	UD		21	106.3					2.07			2.75	Fat Clay w/Sand (CH)
	3	4.0	6.0	UD		23									2.00	Fat Clay (CH)
	4	6.0	8.0	UD		23									2.50	Fat Clay (CH)
	5	8.0	10.0	UD		32		78	28	50	99.2				3.25	Fat Clay (CH)
	6	10.0	12.0	UD		18	114.8						2.08 (7)		3.50	Lean Clay (CL)
	7	12.0	14.0	UD		16									4.50	Lean Clay (CL)
	8	14.0	16.0	UD		21									4.50	Lean Clay (CL)
	9	16.0	18.0	UD		22		60	19	41	92.6				3.75	Fat Clay (CH)
	10	18.0	20.0	UD		26	100.2						1.34 (13)		3.25	Fat Clay (CH)
	11	23.0	25.0	UD		22	104.4						0.79 (16)		3.00	Clayey Sand (SC)
	12	28.0	30.0	UD		30		73	28	45	98.8				4.50	Fat Clay (CH)
	13	33.0	35.0	UD		15									4.50	Fat Clay (CH)
	14	38.0	40.0	UD		17									4.50	Fat Clay (CH)
B-8B	1	0.0	2.0	UD		27									2.25	Fill: Fat Clay (CH)
	2	2.0	4.0	UD		22	105.5	57	19	38	86.6	2.11			2.00	Fat Clay (CH)
	3	4.0	6.0	UD		24									2.50	Fat Clay (CH)
	4	6.0	8.0	UD		33									1.75	Fat Clay (CH)
	5	8.0	10.0	UD		26	99.1						1.10 (6)		3.25	Fat Clay (CH)
	6	10.0	12.0	UD		18		41	13	28	69.7				3.00	Sandy Lean Clay (CL)
	7	12.0	14.0	UD		15									4.50	Sandy Lean Clay (CL)
	8	14.0	16.0	UD		21									4.25	Fat Clay (CH)
	9	16.0	18.0	UD		21	105.9						1.92 (11)		4.50	Fat Clay (CH)
	10	18.0	20.0	UD		24		65	25	40	99.7				4.50	Fat Clay (CH)
	11	23.0	25.0	UD		16	116.8						3.33 (14)		4.25	Lean Clay (CL)
	12	28.0	30.0	UD		16		46	16	30	85.4				4.50	Lean Clay (CL)
	13	33.0	35.0	UD		22	106.2						1.50 (17)		4.50	Lean Clay (CL)
	14	38.0	40.0	UD		21									4.50	Lean Clay (CL)
B-10A	1	0.0	2.0	UD		27	94.5					1.90			2.25	Fat Clay (CH)
	2	2.0	4.0	UD		26		72	22	50	92.7				2.25	Fat Clay (CH)
	3	4.0	6.0	UD		29									2.25	Fat Clay (CH)
	4	6.0	8.0	UD		30	92.1						1.42 (5)		1.75	Fat Clay (CH)
	5	8.0	10.0	UD		15									3.50	Lean Clay w/Sand (CL)
	6	10.0	12.0	UD		18		38	14	24	78.0				3.25	Lean Clay w/Sand (CL)
Legend	UD = UnDisturbed sample, extruded in field SS = Split Spoon sample AG = Auger Cuttings SPT = Standard Penetration Test							LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index UU = Triaxial Compession					Notes:				

SUMMARY OF TEST RESULTS	Project Name: GILETTE TRUNKLINE (GENESEE SEGMENT) PAVING AND DRAINAGE
	WBS Number: M-410290-0003-3
Aviles Engineering Corporation	AEC Project Number: G166-12B

Boring No.	Sample No.	Depth (ft) Top	Depth (ft) Bottom	Type	SPT (blows/ft)	Water Content (%)	Dry Density (pcf)	Atterberg Limits LL (%)	PL (%)	PI (%)	Percent Passing Sieve #200 (%)	Shear Strength (tsf) Unconfined Compression	UU (confining pressure, psi)	Torvane	Pocket Penetrometer	Type of Material
B-10A	7	12.0	14.0	UD		15									4.50	Lean Clay w/Sand (CL)
	8	14.0	16.0	UD		27	103.8						1.65 (10)		1.50	Fat Clay w/Sand (CH)
	9	16.0	18.0	UD		16									3.25	Fat Clay w/Sand (CH)
	10	18.0	20.0	UD		16		75	27	48	79.4				3.25	Fat Clay w/Sand (CH)
	11	23.0	25.0	UD		34	87.3						2.96 (16)		4.00	Fat Clay (CH)
	12	28.0	30.0	UD		27									4.25	Fat Clay (CH)
	13	33.0	35.0	UD		36	86.5	86	35	51	95.3		3.00 (23)		4.25	Fat Clay (CH)
	14	38.0	40.0	UD		14									4.50	Lean Clay (CL)

Legend:

UD = UnDisturbed sample, extruded in field
SS = Split Spoon sample
AG = Auger Cuttings
SPT = Standard Penetration Test

LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
UU = Triaxial Compession

Notes:

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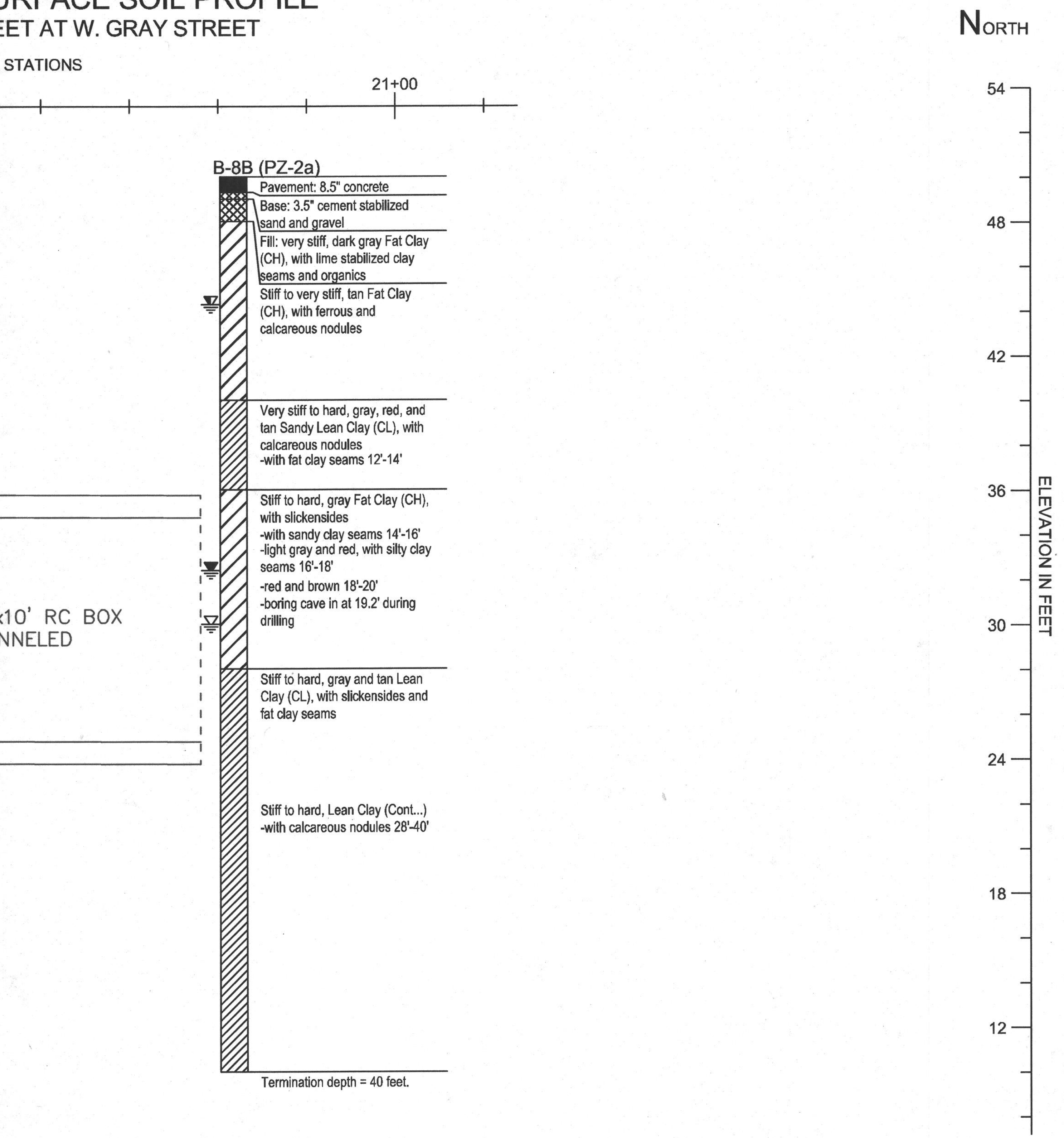
APPENDIX B

Plate B-1	Generalized Soil Profile
Plates B-2 and B-3	Piezometer Installation Details

URFACE SOIL PROFILE

EET AT W. GRAY STREET



secondary soil structure (such as seams, layers, or silts, slickensides, and fissures) that are different from n the actual borings may exist away from these borings.

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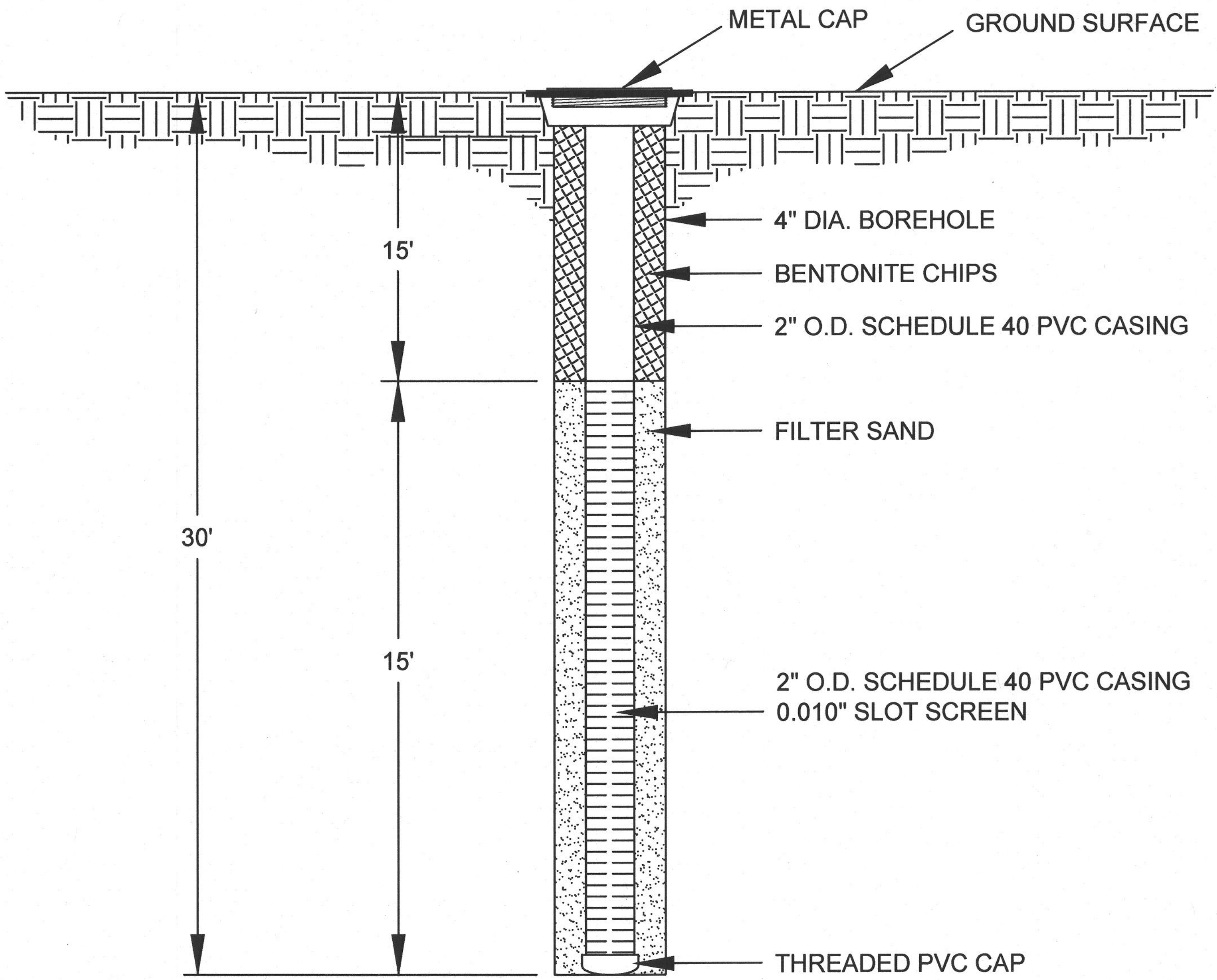
GENERALIZED SOIL PROFILE

GILLETTE TRUNKLINE (GENESEE SEGMENT)

WBS NO. M-410290-0003-3

HOUSTON, TEXAS

AEC PROJECT NO.: G166-12B	DATE: 03-17-15	SOURCE DRAWING PROVIDED BY: AVILES ENGINEERING CORP.
VERTICAL SCALE: 1" = 6' HORIZONTAL SCALE: 1" = 30'	DRAFTED BY: BpJ	PLATE NO.: PLATE B-1



GROUNDWATER DEPTH FROM SURFACE:	DATE MEASURED:
5.7 FT	3/2/15
5.2 FT	3/20/15

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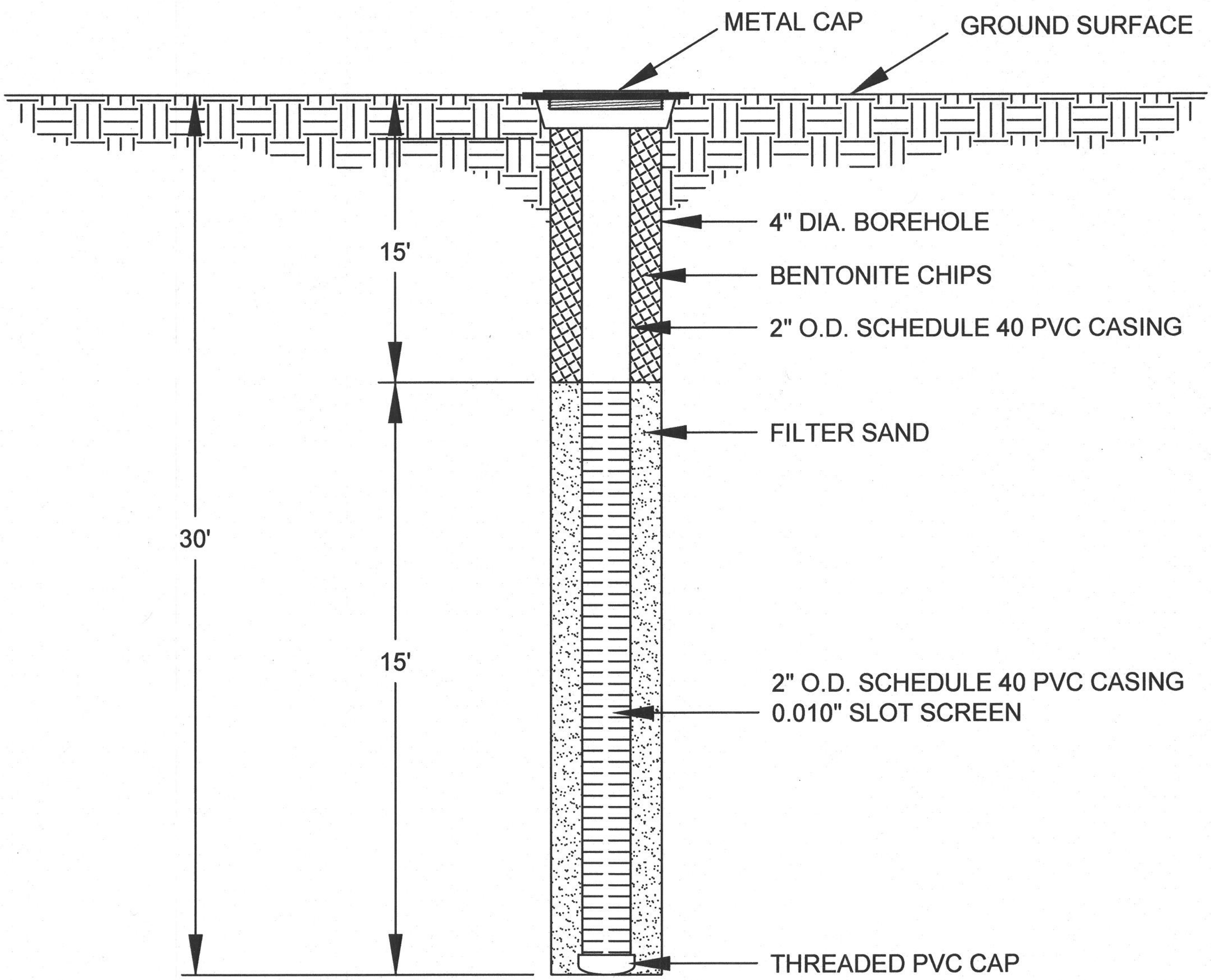
PIEZOMETER INSTALLATION DETAILS
BORING B-8B (PZ-2A)

GILLETTE TRUNKLINE (GENESEE SEGMENT)
WBS NO. M-410290-0003-3
HOUSTON, TEXAS

AEC PROJECT NO.:	DATE:	SOURCE DWG. BY:
G1166-12B	03-02-15	AVILES ENGINEERING CORP.
SCALE: N.T.S.	DRAWN BY: BpJ	PLATE NO.: PLATE B-2



GROUNDWATER DEPTH FROM SURFACE:	DATE MEASURED:
29.3 FT	1/28/15
26.0 FT	3/2/15
5.2 FT	3/20/15

AVILES ENGINEERING CORPORATION

PIEZOMETER INSTALLATION DETAILS
BORING B-10A (PZ-3A)

GILLETTE TRUNKLINE (GENESEE SEGMENT)
WBS NO. M-410290-0003-3
HOUSTON, TEXAS

AEC PROJECT NO.:	DATE:	SOURCE DWG. BY:
G1166-12B	03-02-15	AVILES ENGINEERING CORP.
SCALE: N.T.S.	DRAWN BY: BpJ	PLATE NO.: PLATE B-3

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APPENDIX C

G166-12 GILETTE TRUNKLINE (GENESEE SEGMENT) DRAINAGE AND PAVING IMPROVEMENTS
SOIL PARAMETERS FOR UNDERGROUND UTILITIES

Boring	Depth (ft)	Soil Type	γ (pcf)	γ' (pcf)	OSHA Type	Short-Term					Long-Term				
						C (psf)	φ (deg)	K_a	K_0	K_p	C' (psf)	φ' (deg)	K_a	K_0	K_p
B-8A	0-10	Stiff to very stiff CH	128	66	B	1800	0	1.00	1.00	1.00	175	16	0.57	0.72	1.76
	10-16	Very stiff to hard CL	136	74	B	2100	0	1.00	1.00	1.00	200	18	0.53	0.69	1.89
	16-18	Stiff to very stiff CH	140	78	B	1300	0	1.00	1.00	1.00	125	16	0.57	0.72	1.76
	18-28	SC	126	64	C (18-20)	0	28	0.36	0.53	2.77	0	28	0.36	0.53	2.77
	28-40	Hard CH	125	63	n/a	3600	0	1.00	1.00	1.00	300	16	0.57	0.72	1.76
B-8B	0-2	Fill: very stiff CH	120	58	C	1500	0	1.00	1.00	1.00	150	16	0.57	0.72	1.76
	2-6	Stiff to very stiff CH	129	67	B	1800	0	1.00	1.00	1.00	175	16	0.57	0.72	1.76
	6-10	Stiff to very stiff CH	125	63	B	1100	0	1.00	1.00	1.00	100	16	0.57	0.72	1.76
	10-14	Very stiff to hard CL	120	58	B	2200	0	1.00	1.00	1.00	200	18	0.53	0.69	1.89
	14-17	Stiff to hard CH	128	66	B	1900	0	1.00	1.00	1.00	175	16	0.57	0.72	1.76
	17-22	Stiff to hard CH	128	66	C* (17-20)	1900	0	1.00	1.00	1.00	175	16	0.57	0.72	1.76
	22-32	Very stiff to hard CL	136	74	n/a	3300	0	1.00	1.00	1.00	300	18	0.53	0.69	1.89
	32-40	Stiff to hard CL	129	67	n/a	1500	0	1.00	1.00	1.00	150	18	0.53	0.69	1.89
B-10A	0-4	Stiff to very stiff CH	121	59	B	1900	0	1.00	1.00	1.00	175	16	0.57	0.72	1.76
	4-8	Stiff CH	120	58	B	1600	0	1.00	1.00	1.00	150	16	0.57	0.72	1.76
	8-14	Very stiff to hard CL	120	58	B	2500	0	1.00	1.00	1.00	250	18	0.53	0.69	1.89
	14-22	Stiff to very stiff CH	132	70	B (14-20)	1500	0	1.00	1.00	1.00	150	16	0.57	0.72	1.76
	22-37	Very stiff to hard CH	118	56	n/a	3000	0	1.00	1.00	1.00	300	16	0.57	0.72	1.76
	37-40	Hard CL	120	58	n/a	3000	0	1.00	1.00	1.00	300	18	0.53	0.69	1.89

(1) γ = Unit weight for soil above water level, γ' = Buoyant unit weight for soil below water level.;

(2) C = Soil ultimate cohesion for short term (upper limit of 3,600 psf for design purposes), φ = Soil friction angle for short term;

(3) C' = Soil ultimate cohesion for long term (upper limit of 300 psf for design purposes), φ' = Soil friction angle for long term;

(4) K_a = Coefficient of active earth pressure, K_0 = Coefficient of at-rest earth pressure, K_p = Coefficient of passive earth pressure;

(5) CL = Lean Clay, CH = Fat Clay, SC = Clayey Sand;

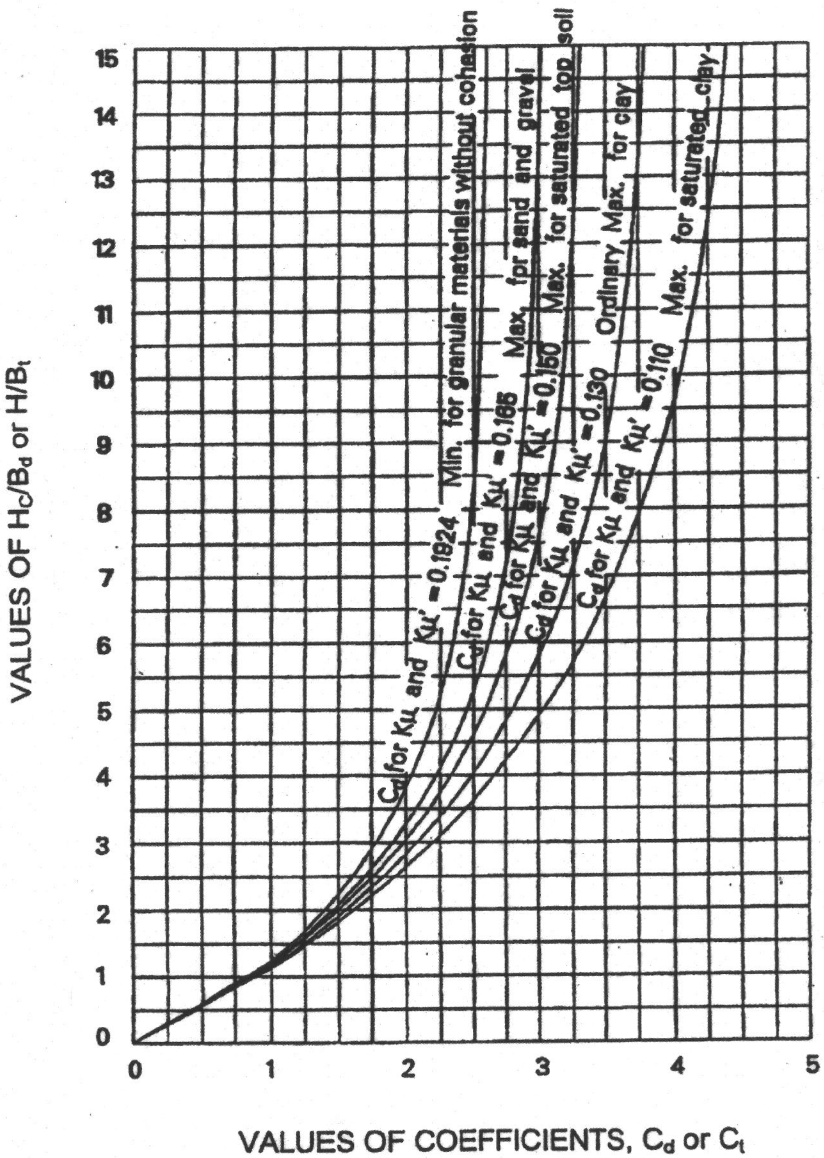
(6) OSHA Soil Types for soils in the top 20 feet below grade:

A: cohesive soils with qu = 1.5 tsf or greater (qu = Unconfined Compressive Strength of the Soil)

B: cohesive soils with qu = 0.5 tsf or greater

C: cohesive soils with qu = less than 0.5 tsf, fill materials, or granular soil

C*: submerged cohesive soils; dewatered cohesive soils can be considered OSHA Type C.



Reference: US Army Corps of Engineers Engineering Manual, EM 1110-2-2902, Oct. 31, 1997, Figure 2-5.

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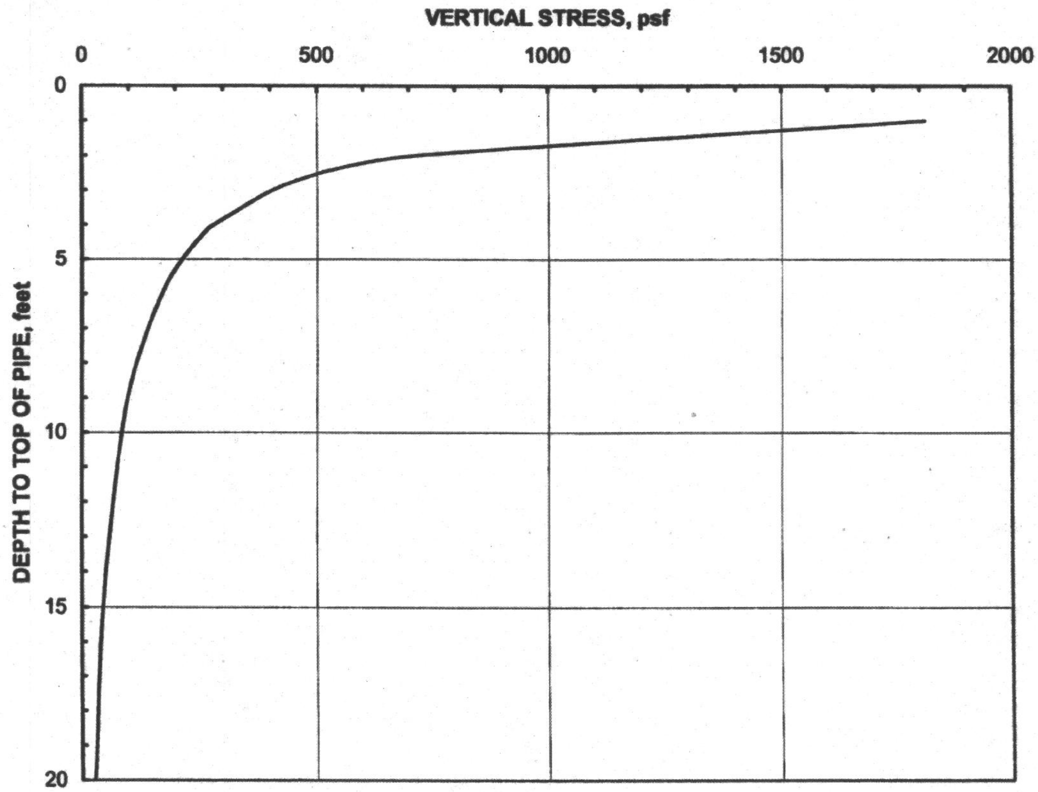

LIVE LOADS ON PIPE CROSSING UNDER ROADWAY

Note: 1. The vertical stress was estimated using AASHTO HS20 truck axle loadings on paved surfaces (Reference: ASCE 15-98, "Standard Practice for Direct Design of Buried Precast Concrete Pipe Using Standandard Installations").

2. Single truck passing.

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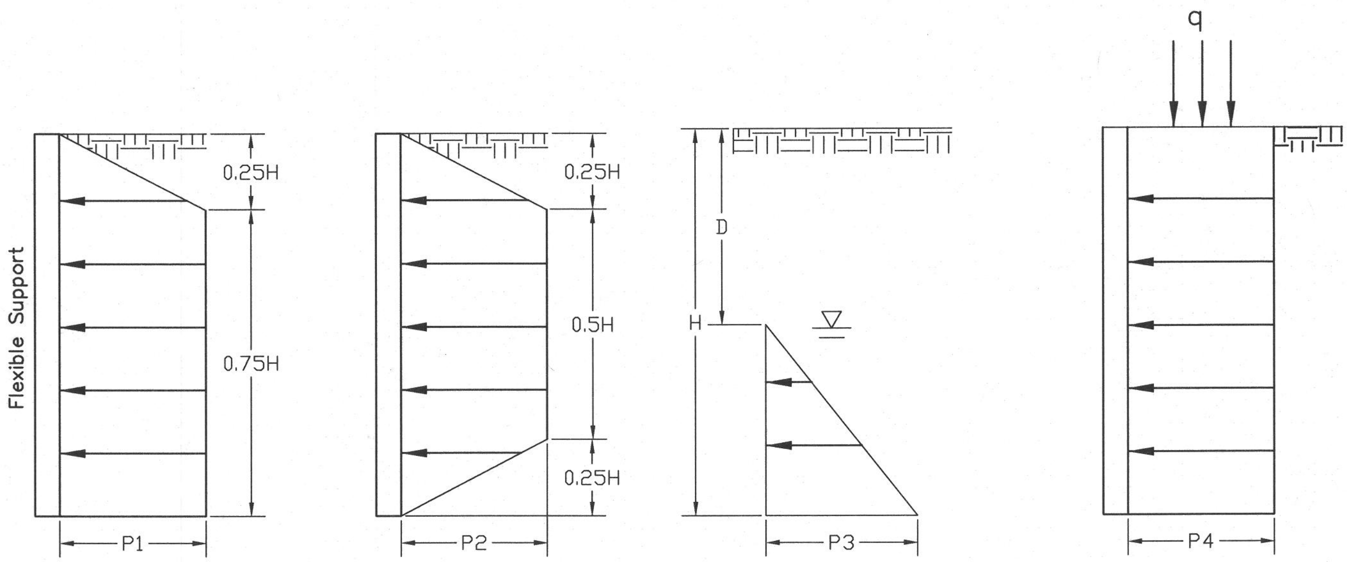
APPENDIX D

LATERAL PRESSURE DIAGRAMS
FOR OPEN CUTS IN COHESIVE SOIL - LONG TERM CONDITIONS



(a) Soft to Medium Clay (b) Stiff Clay (c) Water Pressure (d) Surcharge Pressure

Empirical Pressure Distributions

Where:
H = Total excavation depth, feet
D = Depth to water table, feet
P1 = Lateral earth pressure = γH-4C, psf
P2 = Lateral earth pressure = 0.4γH, psf
P3 = Water pressure = γ_w (H-D), psf
P4 = Lateral earth pressure caused by surcharge = qK_a, psf
γ = Effective unit weight of soil, pcf
γ_w = Unit weight of water, pcf
C = Drained shear strength or cohesion, psf
K_a = Coefficient of active earth pressure

Notes:
1. All pressures are additive.
2. No safety factors are included.
3. For use only during long term construction.
4. If $\gamma H/C < 4$, use section (b),
   If $4 < \gamma H/C < 6$, use larger of section (a) or (b),
   If $\gamma H/C > 6$, use section (a).

Reference: Peck, R.B. (1969), "Deep Excavation and Tunneling in soft Ground", 7th ICSMFE, State of art volume, pp. 225-290.

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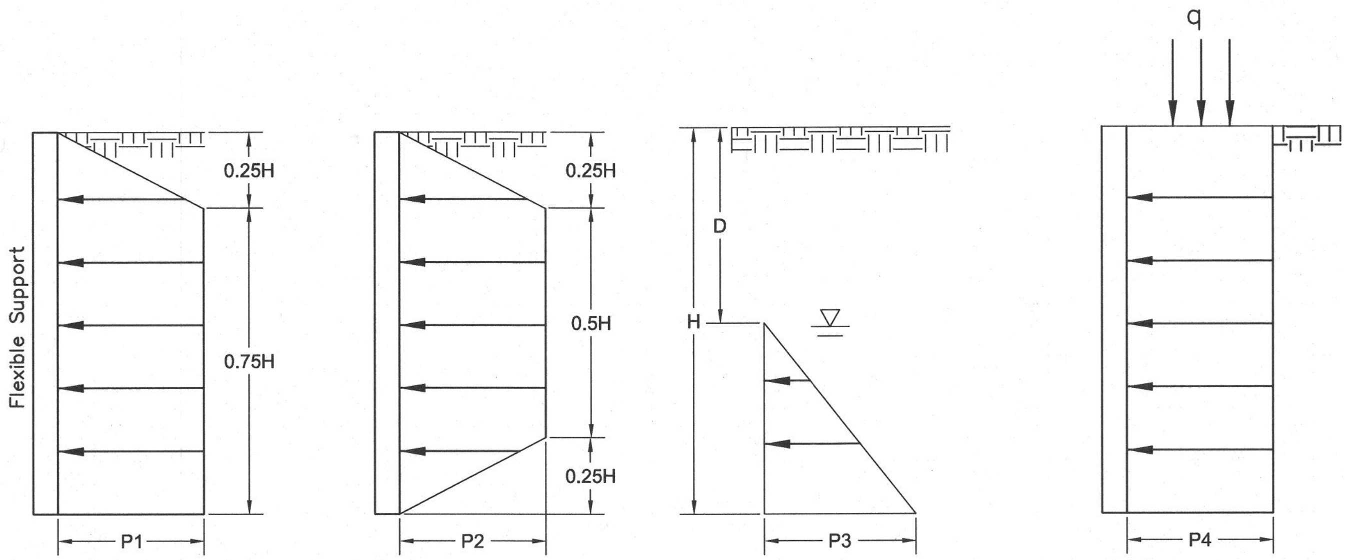
LATERAL PRESSURE DIAGRAMS
FOR OPEN CUTS IN COHESIVE SOIL - SHORT TERM CONDITIONS



(a) Soft to Medium Clay

(b) Stiff Clay

(c) Water Pressure

(d) Surcharge Pressure

Empirical Pressure Distributions

Where:
H = Total excavation depth, feet
D = Depth to water table, feet
P1 = Lateral earth pressure = $\gamma H-4S_u$, psf
P2 = Lateral earth pressure = $0.2\gamma H$, psf
P3 = Water pressure = γ_w (H-D), psf
P4 = Lateral earth pressure caused by surcharge = qK_a, psf
γ = Effective unit weight of soil, pcf
γ_w = Unit weight of water, pcf
S_u = Undrained shear strength = $q_u/2$, psf
q_u = Unconfined compressive strength, psf
K_a = Coefficient of active earth pressure

Notes:
1. All pressures are additive.
2. No safety factors are included.
3. For use only during short term construction.
4. If $\gamma H/S_u < 4$, use section (b),
   If $4 < \gamma H/S_u < 6$, use larger of section (a) or (b),
   If $\gamma H/S_u > 6$, use section (a).

Reference: Peck, R.B. (1969), "Deep Excavation and Tunneling in soft Ground", 7th ICSMFE, State of art volume, pp. 225-290.

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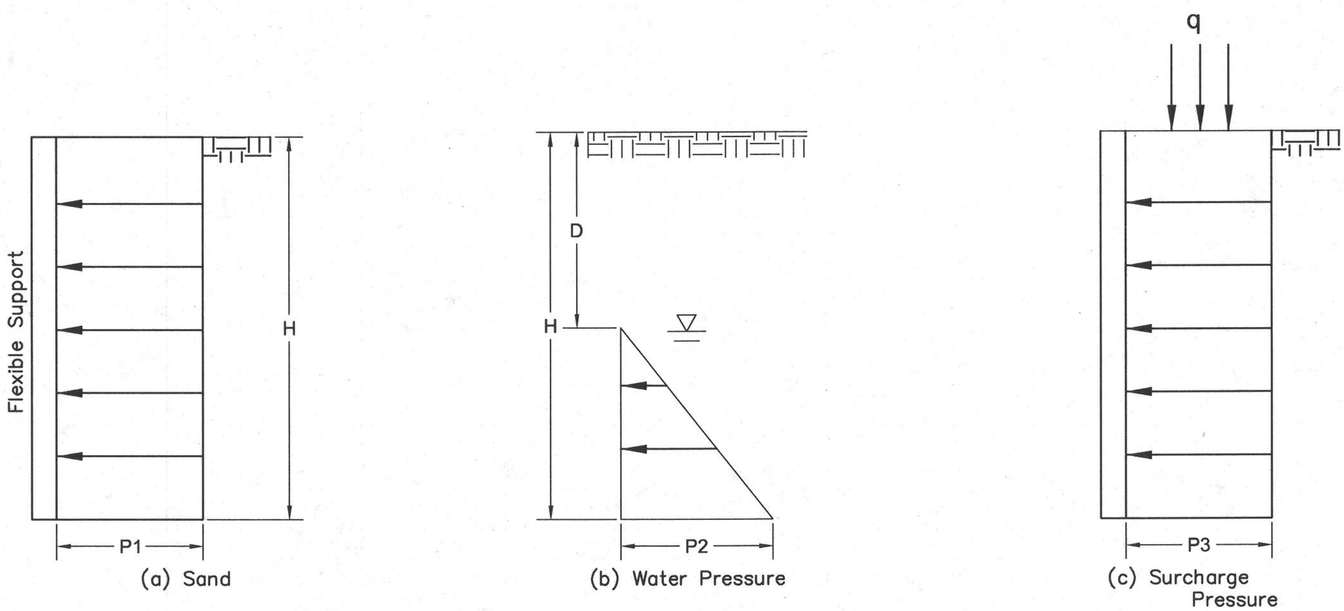
LATERAL PRESSURE DIAGRAMS
FOR OPEN CUTS IN SAND



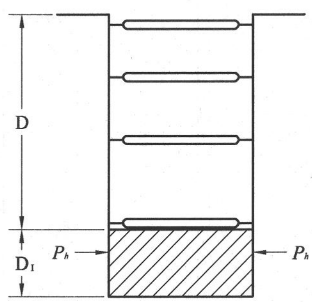
Empirical Pressure Distributions

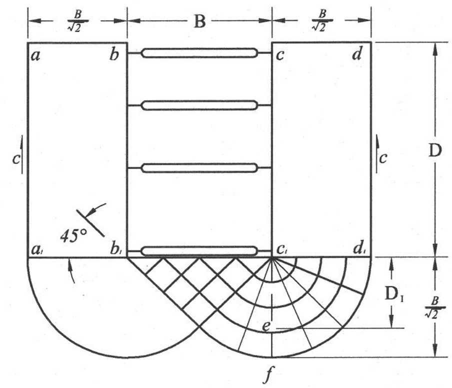
Where:
H = Total excavation depth, feet
D = Depth to water table, feet
P1 = Lateral earth pressure = $0.65*\gamma HK_a$, psf
P2 = Water pressure = γ_w (H-D), psf
P3 = Lateral earth pressure caused by surcharge = qK_a, psf
γ = Effective unit weight of soil, pcf
γ_w = Unit weight of water, pcf
K_a = Coefficient of active earth pressure = $(1-\sin\varphi)/(1+\sin\varphi)$
φ = Drained friction angle

Notes:
1. All pressures are additive.
2. No safety factors are included.

Reference: Peck, R.B. (1969), "Deep Excavation and Tunneling in soft Ground", 7th ICSMFE, State of art volume, pp. 225-290.

BOTTOM STABILITY FOR BRACED EXCAVATION IN CLAY





Factor of Safety against bottom of heave,

$$F.S = \frac{NcC}{(\gamma D+q)}$$

where, Nc = Coefficient depending on the dimension of the excavation (see Figure at the bottom)
C = Undrained shear strength of soil in zone immediately around the bottom of the excavation,
γ = Unit weight of soil,
D = Depth of excavation,
q = Surface surcharge.

If F.S < 1.5, sheeting should be extended further down to achieve stability

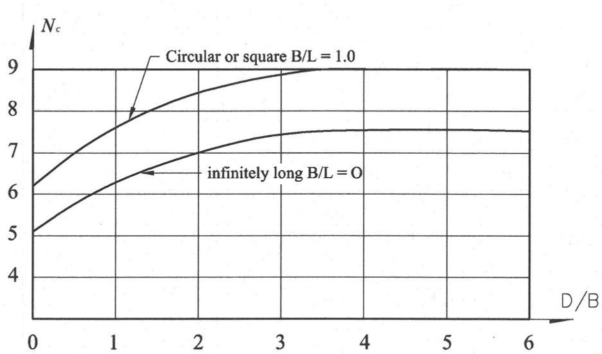
$$\textit{Depth of Buried Length, } (D_l) = \frac{1.5(\gamma D+q)-NcC}{(C/B)-0.5\gamma} \; ; D_l \geq 5\,ft.$$

Pressure on buried length, P_h:

For $D_l < 0.47B$; $P_h = 1.5\, D_l(\gamma D - 1.4\, CD/B - 3.14C)$

For $D_l > 0.47B$; $P_h = 0.7\, (\gamma DB - 1.4\, CD - 3.14CB)$

where; B = width of excavation



N_c rectangular = $(0.84 + 0.16B/L)N_c$ square

Reference: Bjerrum, L. and Eide, O., Stability of Strutted Excavations in Clay, Geotechnique, 6, 32-47 (1956).

Tunnel Behavior: Sands and Gravels
(Terzaghi, 1977)

Designation	Degree of Compactness	Tunnel Behavior	
		Above Water Table	Below Water Table
Very Fine Clean Sand	Loose, $N \leq 10$ Dense, $N > 30$	Cohesive Running Fast Raveling	Flowing Flowing
Fine Sand with Clay Binder	Loose, $N \leq 10$ Dense, $N > 30$	Rapid Raveling Firm or Slowly Raveling	Flowing Slowly Raveling
Sand or Sandy Gravel with Clay Binder	Loose, $N < 10$ Dense, $N > 30$	Rapid Raveling Firm	Rapidly Raveling or Flowing Firm/slow Raveling
Sandy Gravel and Medium to Coarse Sand		Running Ground. Uniform ($C_u < 3$) and loose ($N < 10$) materials with round grains run much more freely than well graded ($C_u > 6$) and dense ($N > 30$) ones with angular grains.	Flowing Conditions combined with extremely heavy discharge of water.

TBM FAMILY OF MACHINES (From Kessler & Moore,)		
Machine Type	**Typical Machine Diameters**	**Ground Condition TBM is Best Suited For**
Pipe Jacking Machines	Up to approx. 10 – 13 ft (3 - 4m)	Any ground
Small Bore Unit (SBU)	Up to 6.6 ft (2m)	Any ground
Shielded TBMs	6.6 – 46 ft (2 to 14m) plus	Soft ground above the water table
Mix Face TBMs	6.6 – 46 ft (2 to 14m) plus	Mixed ground above the water table
Slurry TBMs	6.6 – 46 ft (2 to 14m) plus	Coarse-grained soft ground below the water table
EPB TBMs	6.6 – 46 ft (2 to 14m) plus	Fine-grained soft ground below the water table
Hard Rock TBMs	6.6 – 46 ft (2 to 14m) plus	Hard rock
Reamer TBMs	Various	Hard rock
Multi-head TBMs	Various	Various

Reference: Dots Oyenuga (2004), "FHWA Road Tunnel Design Guidelines", Pages 8 and 10, published by U.S. Department of Transportation Federal Highway Administration, Report No. FHWA-IF-05-023, Washington DC.

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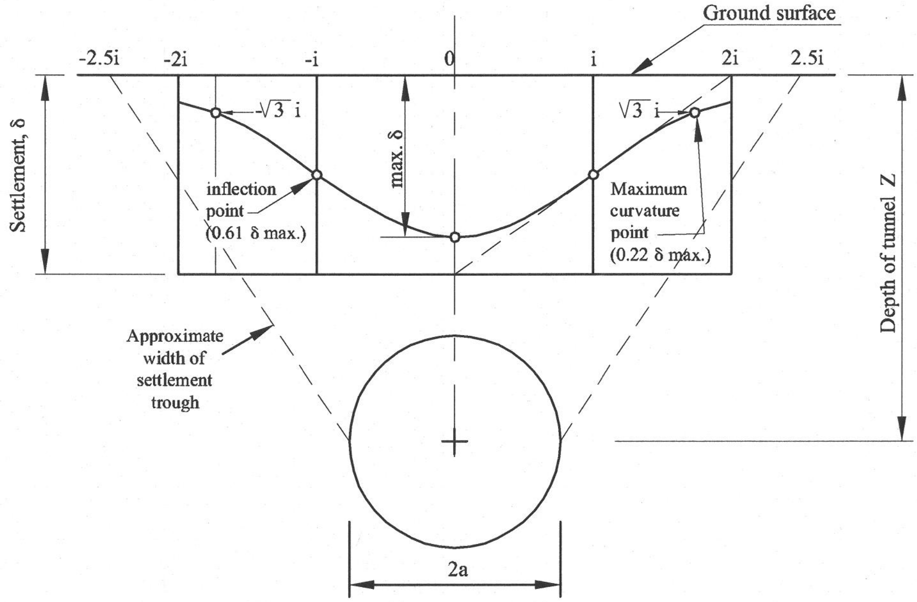
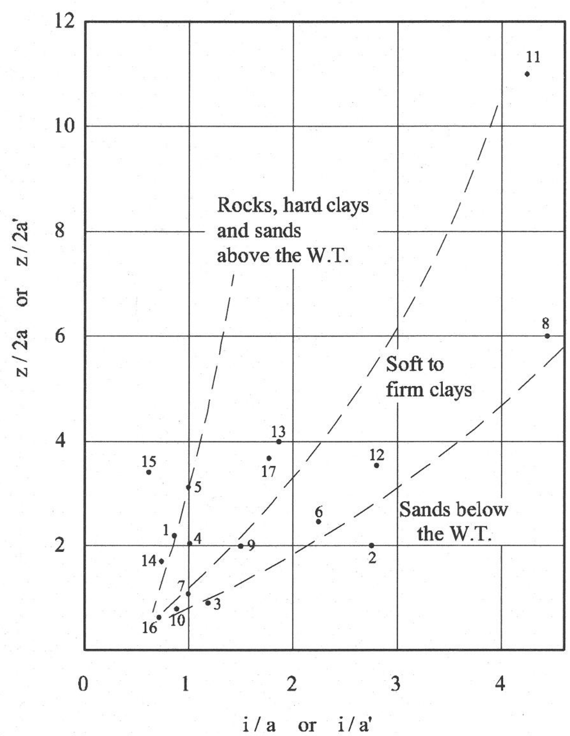
Relation between the Width of the Surface Depression (i/a) and the Depth of the Cavity (z/a) for Tunnels



Volume of depression = 2.5i δ max.

(a)

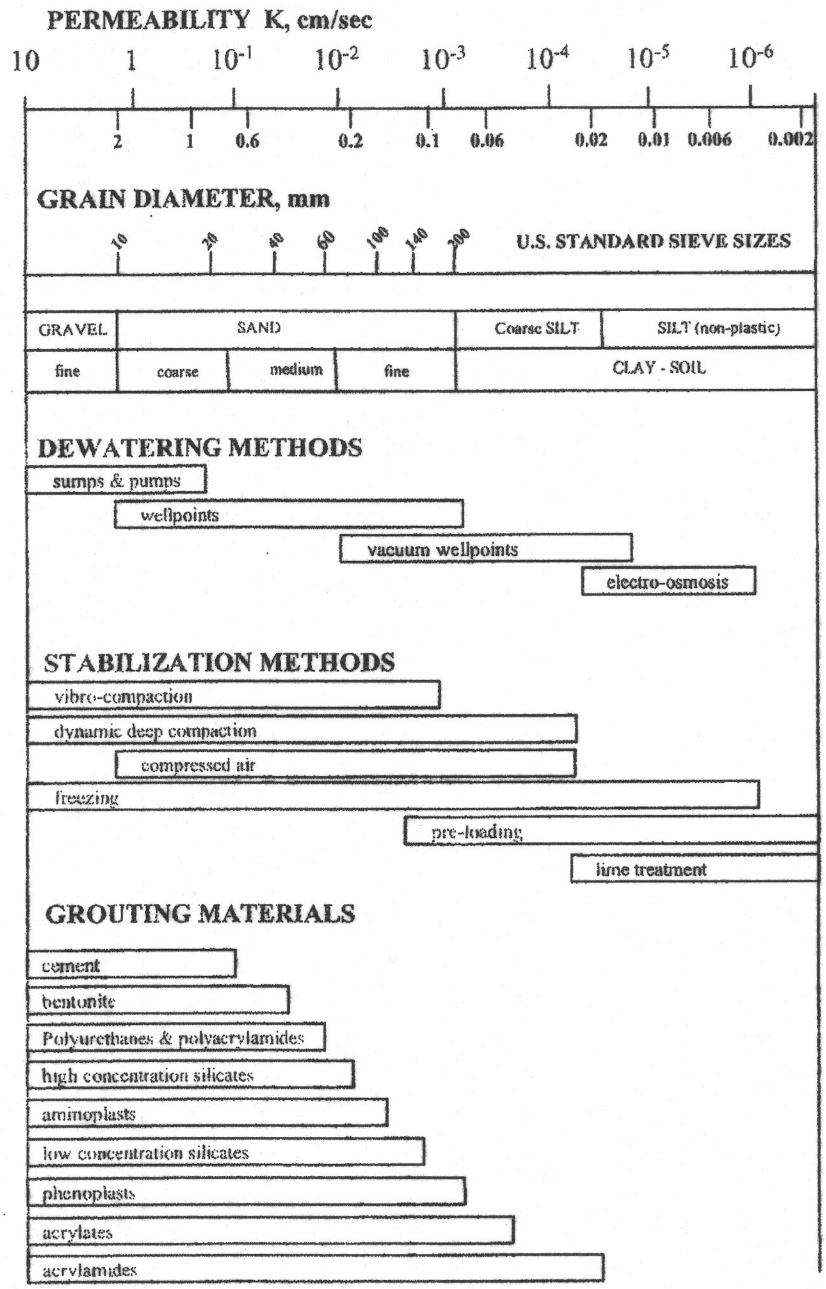


(b)

Reference: Peck, R. B. (1969) "Deep Excavations and Tunneling in Soft Ground," Proceedings, Seventh International Conference on Soil Mechanics and Foundation Engineering, Mexico City, State of the Art Volume, pp. 225-290.

AVILES
ENGINEERING CORP.

Methods of Controlling Groundwater
(after Karol, 1990)



Note: 1 cm/sec = 0.4 in/sec; 1 mm = 0.04 in.

Reference: Dots Oyenuga (2004), "FHWA Road Tunnel Design Guidelines", Page 9, published by U.S. Department of Transportation Federal Highway Administration, Report No. FHWA-IF-05-023, Washington DC.

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APPENDIX E

Plates E-1 to E-4 Piezometer Installation and Plugging Reports

STATE OF TEXAS WELL REPORT for Tracking #390939

Owner:	**City of Houston Geotechnical Dept**	Owner Well #:	**pz-2A**
Address:	**611 Walker Floor 14** **Houston , TX 77002**	Grid #:	**65-13-9**
Well Location:	**107 Welch St** **1600 Genessee , TX 77019**	Latitude:	**29° 45' 12" N**
Well County:	**Harris**	Longitude:	**095° 23' 02" W**
Elevation:	**No Data**	GPS Brand Used:	**No Data**
Type of Work:	**New Well**	Proposed Use:	**Monitor**

Drilling Date:	Started: **1/27/2015** Completed: **1/27/2015**
Diameter of Hole:	Diameter: **4 in From Surface To 30 ft**
Drilling Method:	**Mud Rotary**
Borehole Completion:	Other: **(No Data)**
Annular Seal Data:	1st Interval: **From 0 ft to 16 ft with 1 cement (#sacks and material)** 2nd Interval: **From 16 ft to 18 ft with .5 bentonite (#sacks and material)** 3rd Interval: **No Data** Method Used: **No Data** Cemented By: **No Data** Distance to Septic Field or other Concentrated Contamination: **No Data** Distance to Property Line: **No Data** Method of Verification: **No Data** Approved by Variance: **No Data**
Surface Completion:	**Surface Sleeve Installed**
Water Level:	Static level: **5'2" ft. below land surface on 3/20/2015** Artesian flow: **No Data**
Packers:	**20/40 18-30**
Plugging Info:	Casing left in well: Cement/Bentonite left in well: From (ft) To (ft) From (ft) To (ft) Cem/Bent Sacks Used **30' of 2" pvc well material was grouted in place via tremmie method on 03/20/2015 by Christopher Olvera**
Type Of Pump:	**No Data**
Well Tests:	**No Data**
Water Quality:	Type of Water: **No Data** Depth of Strata: **No Data** Chemical Analysis Made: **No** Did the driller knowingly penetrate any strata which contained undesirable constituents: **No**
Certification Data:	The driller certified that the driller drilled this well (or the well was drilled under the driller's direct supervision) and that each and all of the statements herein are true and correct. The driller understood that failure to complete the required items will result in the log(s) being returned for completion and resubmittal.
Company Information:	

Van and Sons Drilling Service
319 John Alber
Houston , TX 77076

Driller License Number:	3286
Licensed Well Driller Signature:	Mark Thornton
Registered Driller Apprentice Signature:	Christopher Olvera
Apprentice Registration Number:	No Data
Comments:	Wells were set by Mark Thornton and plugged by Chris Olvera

IMPORTANT NOTICE FOR PERSONS HAVING WELLS DRILLED CONCERNING CONFIDENTIALITY

TEX. OCC. CODE Title 12, Chapter 1901.251, authorizes the owner (owner or the person for whom the well was drilled) to keep information in Well Reports confidential. The Department shall hold the contents of the well log confidential and not a matter of public record if it receives, by certified mail, a written request to do so from the owner.

Please include the report's Tracking number (Tracking #390939) on your written request.

Texas Department of Licensing & Regulation
P.O. Box 12157
Austin, TX 78711
(512) 463-7880

DESC. & COLOR OF FORMATION MATERIAL

From (ft) To (ft) Description
na

CASING, BLANK PIPE & WELL SCREEN DATA

Dia. New/Used Type Setting From/To
2 n sch 40 pvc riser 0-20
2 n sch 40 pvc screen 20-30 .010

STATE OF TEXAS WELL REPORT for Tracking #390938

Owner:	**City of Houston Geotechnical Dept**	Owner Well #:	**pz-3A**
Address:	**611 Walker Floor 14** **Houston , TX 77002**	Grid #:	**65-13-9**
Well Location:	**107 Welch St** **Houston , TX 77006**	Latitude:	**29° 45' 01" N**
Well County:	**Harris**	Longitude:	**095° 23' 03" W**
Elevation:	**No Data**	GPS Brand Used:	**No Data**
Type of Work:	**New Well**	Proposed Use:	**Monitor**

Drilling Date:	Started: **1/27/2015** Completed: **1/27/2015**
Diameter of Hole:	Diameter: **4 in From Surface To 30 ft**
Drilling Method:	**Mud Rotary**
Borehole Completion:	Other: **(No Data)**
Annular Seal Data:	1st Interval: **From 0 ft to 16 ft with 1 cement (#sacks and material)** 2nd Interval: **From 16 ft to 18 ft with .5 bentonite (#sacks and material)** 3rd Interval: **No Data** Method Used: **No Data** Cemented By: **No Data** Distance to Septic Field or other Concentrated Contamination: **No Data** Distance to Property Line: **No Data** Method of Verification: **No Data** Approved by Variance: **No Data**
Surface Completion:	**Surface Sleeve Installed**
Water Level:	Static level: **5'2" ft. below land surface on 3/20/2015** Artesian flow: **No Data**
Packers:	**20/40 18-30**
Plugging Info:	Casing left in well: Cement/Bentonite left in well: From (ft) To (ft) From (ft) To (ft) Cem/Bent Sacks Used **30' of 2" pvc well material was grouted in place via tremmie method on 03/20/2015 by Christopher Olvera**
Type Of Pump:	**No Data**
Well Tests:	**No Data**
Water Quality:	Type of Water: **No Data** Depth of Strata: **No Data** Chemical Analysis Made: **No** Did the driller knowingly penetrate any strata which contained undesirable constituents: **No**
Certification Data:	The driller certified that the driller drilled this well (or the well was drilled under the driller's direct supervision) and that each and all of the statements herein are true and correct. The driller understood that failure to complete the required items will result in the log(s) being returned for completion and resubmittal.
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P.O. Box 12157
Austin, TX 78711
(512) 463-7880

DESC. & COLOR OF FORMATION MATERIAL

From (ft) To (ft) Description
na

CASING, BLANK PIPE & WELL SCREEN DATA

Dia. New/Used Type Setting From/To
2 n sch 40 pvc riser 0-20
2 n sch 40 pvc screen 20-30 .010